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AIR TRANSPORT DIVISION

Proceedings of the American Society of Civil Engineers

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Papers

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Journal of the
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REINFORCED CONCRETE PAVEMENTS FOR AIRPORTS[a]

By August W. Compton,[1] F. ASCE

SYNOPSIS

The use of properly distributed wire reinforcement in concrete airfield pavements controls cracking, reduces maintenance, and increases service life. As an example of the design method that is proposed, the pavement of the Indianapolis, Ind., Municipal Airport is described.

INTRODUCTION

Distributed steel reinforcement has been in general use in concrete pavements for more than 50 yr. Nevertheless, concrete pavements are still being constructed without reinforcement. Some constructing authorities feel that the additional cost is not warranted or that traffic conditions do not require it. However, all agree that it is beneficial.

In 1925, after a Highway Research Board (HRB) survey of some 5,500 miles of road, of which 2,000 miles were inspected in detail, C. A. Hogentogler concluded[2] that (1) the amount of cracking and subsequent disintegration is a function of time; thus the rate of cracking is a measure of the life of the pavement, and (2) that steel reinforcement reduces the rate of cracking and thus increases the life of the pavement.

Note.—Discussion open until October 1, 1960. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Air Transport Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. AT 1, May, 1960.

[a] Presented at the October 1959 ASCE Convention in Washington, D.C.

[1] Vice Pres., Clyde E. Williams and Assocs., Inc., Indianapolis, Ind.

[2] "Economic Value of Reinforcement in Concrete Roads," by C. A. Hogentogler, Proceedings of the 5th Annual Meeting of the Highway Research Board, Part II.

For a number of years, highway engineers have tried to control cracking in non-reinforced pavements by forming or sawing planes of weakness in the slab at 15 ft to 20 ft spacings. These slabs depend on aggregate interlock to provide load transfer from one slab to another. With the increase in both volume and weight of trucks, the performance of these slabs has not been satisfactory. As a result, many state highway departments have returned to a reinforced design with slabs 40 ft to 100 ft long, providing mechanical load transfer devices at the joints and distributed steel reinforcement between the joints.

Both types of design have been used and are now being used for airport pavements. Reasons for using or not using reinforcement are the same as those for highways. Of course, they should be. The volume of traffic on airport pavements is much less than on highways but the wheel loads are many times greater.

AIRPORT PAVEMENT RESEARCH

In general, it has been considered that steel reinforcement, in the amounts normally used in concrete pavements, does not add to the structural strength of the slab. However, it must be remembered that a reinforced concrete design provides adequate mechanical load transfer at joints and sufficient distributed steel to hold tightly closed any cracks that may form in between the joints. In contrast, a crack or a preformed crack such as one produced by a plane-of-weakness type of joint in a non-reinforced pavement, results in two separate slabs divided by an irregular opening of changing width. Such an open crack is a point of stress concentration, increased deflection, and subgrade deterioration. On the other hand, at two adjacent slabs in a reinforced pavement, whether at a crack or at a joint, the wheel loads are supported by both slabs acting together rather than by each slab in turn. Reduced deflections and flexural stresses resulting from this slab action are, in themselves, structural benefits.

In the final report[3] of Lockbourne No. 1 Test Track, March, 1946, the Corps of Engineers concluded:

1. Steel reinforcement in concrete pavements prolongs their serviceable life under traffic loading, especially when pavements become overloaded.
2. The benefits resulting from the inclusion of reinforcement in overloaded concrete pavements increase with the amount of steel present.
3. When the overload becomes excessive the effect of different amounts of reinforcement is lost, and the steel also loses some of its power to prolong pavement life.
4. Assuming that subgrade conditions were equal for both reinforced and unreinforced slabs, results of stationary wheel load deflection measurements indicated that reinforcing was effective in reducing the magnitude of pavement deflections at points of overload such as free edges and corners.

Frank M. Mellinger, F. ASCE James P. Sale, M. ASCE, and Thurman R. Wathen, A.M. ASCE, have reported[4] the results of accelerated traffic tests on

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[3] Final Report—Lockbourne No. 1 Test Track, Corps of Engrs., U.S. Army, March, 1946.

[4] "Heavy Wheel Load Traffic on Concrete Airfield Pavements," by Frank M. Mellinger, James P. Sale and Thurman R. Wathen, Proceedings of the 36th Annual Meeting of the Highway Research Board.

various thickness of reinforced and nonreinforced concrete pavements subjected to a load of 100,000 lb on a dual wheel. They concluded, with respect to steel reinforcing, that:

a. Reinforcing does not materially effect the number of load repetitions required to produce the first crack in a concrete pavement.

b. The rate of progression of cracking after an initial crack is much slower in reinforced pavement than in non-reinforced pavement.

c. The cracks which develop in reinforced pavement are held tightly together.

d. Nominal amounts of reinforcement in concrete pavements increase their useful life and may be used to reduce the thickness of concrete within the limitations given in the preceding discussion.

e. There is no advantage in placing reinforcement in the amounts considered near the bottom of a pavement slab.

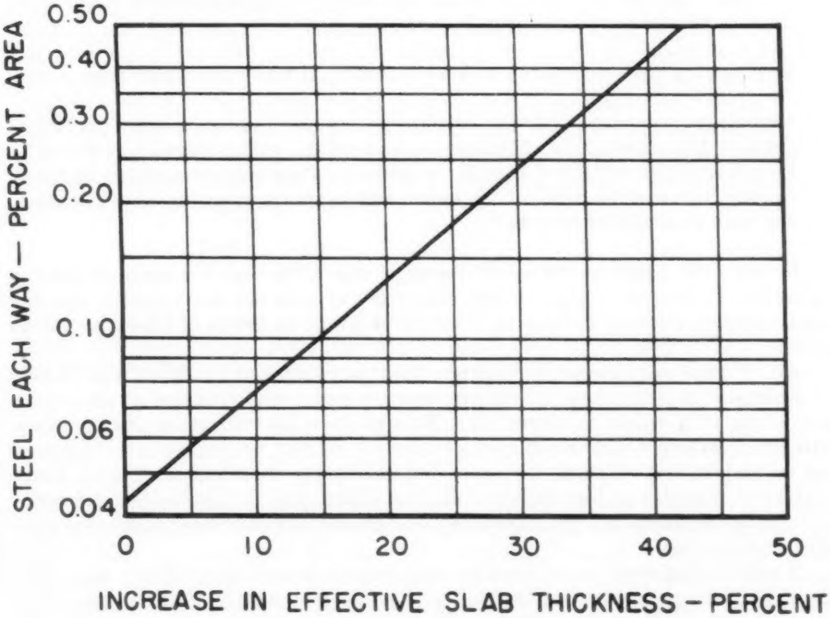

FIG. 1.—EFFECT OF STEEL REINFORCEMENT ON RIGID PAVEMENTS

These findings demonstrate conclusively that the addition of reinforcing steel in a concrete pavement is a distinct structural advantage which is reflected in a longer service life. Also, it is especially interesting to observe that the basic conclusions from the 1946 and 1957 research on airport pavements are almost identical with those presented in 1925 by Hogentogler as a result of his studies of highway pavements. This certainly cannot be an accident or a coincidence. It is very definitely a basic performance characteristic.

Mellinger, Sale and Wathen include, in the report, a curve showing the increase in effective slab thickness resulting from the use of different percentages of reinforcing steel. This curve, Fig. 1, which the authors appear to con-

sider very conservative, shows an appreciable increase in structural strength when the pavement is reinforced. Design curves, based on these data, are used by the Corps of Engineers as a basis for reducing pavement thickness when the slabs are reinforced.

In 1948 a Civil Aeronautics Administration publication allowed a 1-in. reduction in pavement thickness when reinforcing steel was included.[5] Based on the previously mentioned Lockbourne Tests it was stated further:

> "Since these tests were made with both static and moving wheel loads of 37,000 and 60,000 pounds, the conclusions suggest that it would be desirable to reinforce concrete pavements in such critical load areas as taxiways, aprons, and runway ends when single wheel loads are expected to exceed 37,000 pounds, or rather in pavements designed to carry single wheel loads of 37,000 pounds or more, especially when overloading with respect to type and quantity of traffic may be anticipated. In these cases, the required amount of reinforcing should be incorporated without a reduction in pavement thickness."

Recognizing the importance of load transfer, it was stated also that

> "dowels are required across all dummy joints in aprons, taxiways and critical thickened ends of runways, and on the entire runway designed for wheel loads of 50,000 pounds or greater. They may be omitted in the runway between the critical thickened end sections when the wheel loads are less than 60,000 pounds."

In the 1956 issue of "Airport Paving," the CAA does not recommend any reduction in thickness when reinforcing steel is used but continues to call for load-transfer devices across all transverse joints in areas of load concentration.

All of these requirements, together with the results of research and studies of pavement performance, reveal the importance of load transfer at all cracks and joints in a concrete pavement. Progressive deterioration and breakage will occur at any crack which opens to the extent that aggregate interlock cannot be maintained. Unless the crack is held tightly closed at all times, there will be no transfer of load between the two slabs. The only economical method of holding the slabs in tight interlock is by means of reinforcing steel of proper size and spacing.

If reinforcing steel is not used the concrete pavement must be cut into slabs about 20 ft long. In areas of traffic concentration as well as areas subjected to heavy wheel loadings the joints at 20-ft spacing should be provided with mechanical load-transfer devices. Under these conditions it seems more logical to reinforce the pavement so that the slabs can be lengthened and the number of joints reduced. The reduction in joints requiring load-transfer devices leads to savings that in many cases will offset the initial cost of the steel reinforcing. Also, future savings in maintenance can be anticipated because the pavement is more sound structurally and there are less joints to maintain.

As an example, at the Municipal Airport at Indianapolis, Ind., during the period of 1930-1934, the city constructed a Northeast-Southwest runway of reinforced concrete. The design of this pavement was based on the aircraft which were the design transports of that period, that is, the DC-2 and the DC-3. In

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[5] "Airport Paving," Civ. Aeronautics Admin., May, 1948 and October, 1956.

1950, most of this early pavement, some 8-6-8 and some 6 in. uniform slab, had by this time reached the apparent end of its useful life. During the final 8 yr of this period, the pavement had been under heavy use by aircraft not in existence when the pavement was designed and built.

Even though the crack pattern in the pavement had reached an advanced stage in some portions, the wire fabric reinforcement held the slabs together sufficiently to permit operations to continue. Obviously this would not have been possible with unreinforced concrete pavement segments of comparable size.

During the 1950 construction season, after a complete pavement evaluation, about 20% of the old pavement was replaced with heavier wire reinforced slabs of thickness equal to the surrounding pavement—the balance was determined to be suitable to serve if a bituminous overlay were added. After replacement of the worst damaged sections, the entire runway was overlain with 6 in. of bituminous overlay material and continued in service.

This pavement is serving (1960) as a part of one of the main taxiways at this airport and is under continuous use by heavy transports. An inspection of this taxiway indicated need for renewal of the wearing surface to improve non-skid qualities but no further structural improvement of the pavement was deemed necessary.

It is the writer's estimate that this airport was able to have about 4 or 5 additional years use from this pavement because of the wire reinforcement before rebuilding was required. There are probably some additional benefits in the amount of pavement that was retained in service; this, however, would be very difficult to evaluate.

COMPUTATION OF STEEL AREA

Determination of the cross-sectional area of steel required for pavement slabs of variable length and thickness is based on the "subgrade drag" theory as expressed by

$$A_s = \frac{F\,L\,w}{2\,f_s} \quad \dots\dots\dots\dots\dots\dots \quad (1)$$

in which A_s is the cross-sectional area of steel required per foot of width of slab, F denotes the coefficient of subgrade friction, w is the weight of slab per square foot, f_s represents the allowable working stress in steel, and L is the length of slab in feet. In Eq. 1 the coefficient of subgrade friction (F) is taken as a variable depending on the length and thickness of the slab. The weight of slab (w) is calculated on a basis of 12.5 lb per sq ft per in. of thickness.

The allowable working stress in the steel depends on its type and grade; this stress should be no greater than 2/3 of the minimum yield strength.[6] On this basis, the American Concrete Institute (ACI) recommends the values in Table 1.

Since this publication was issued, the minimum yield point of cold drawn wire was raised to 60,000 psi. Accordingly, the allowable stress (f_s), on the same basis, would be 40,000 psi. Observations of pavements in service, however, indicate that a design stress of 45,000 psi will not be excessive with this material. The writer has used 45,000 psi as design stress in computations for

6 "Recommended Practice for Design of Concrete Pavements," (ACI 325-58), Amer. Concrete Inst.

several airfield pavement projects and observed their performance for several years of heavy service. Even though some of these pavements have been subjected to loads considerably over design, their performance is normal in every respect and seems to justify the designer's decision.

COEFFICIENT OF SUBGRADE

Tests with relatively thin slabs have shown that the coefficient of subgrade friction may vary from 1.0 to 2.0. An average value of 1.5 is customarily used for highway pavements. For the thicker airport pavements, however, it seems desirable to take advantage of the fact that the coefficient of friction is variable and tends to decrease as the slab thickness is increased, but, at the same time, becomes greater as slabs become longer.

TABLE 1

Type and Grade of Steel	Minimum Yield Point, in psi	f_s, in psi
Billet and axle steel, structural grade	33,000	22,000
Billet and axle steel, intermediate grade	40,000	27,000
Rail steel or billet and axle steel, hard grade	50,000	33,000
Cold drawn wire	56,000	37,000

For slabs between 40 ft and 100 ft in length, the Wire Reinforcement Institute recommends that the coefficient of friction be determined as suggested by E. F. Kelley.[7] This method is also the basis for the steel design formulas in the CAA manual "Airport Paving." Calculations by this method show that for slabs 70 ft to 80 ft long the coefficient of subgrade friction varies approximately in accordinace with the formula.

$$F = 0.55\sqrt{\frac{L}{h}} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (2)$$

in which h is the slab thickness, in inches.

REINFORCEMENT DESIGN, MUNICIPAL AIRPORT AT INDIANAPOLIS

The foregoing criteria were used in designing runway, apron, and taxiway pavements for the Municipal Airport at Indianapolis. Three different pavement thicknesses were involved; 8 in., 10 in., and 12 in. The length of slab in all instances was standardized at 75 ft and the allowable working stress in the steel (welded wire fabric) was taken as 45,000 psi.

Values for the coefficient of subgrade friction, F, were 1.7, 1.5, and 1.4, respectively, for the slabs that were 8 in., 10 in., and 12 in. thick, respectively.

[7] "Applications of the Results of Research to the Structural Design of Pavements," by E. F. Kelley, Public Roads, August, 1939.

Substituting these values in Eq. 2 and using L = 75 ft for the longitudinal steel area and L = 25 ft for the area of the transverse steel, the required steel areas were as shown in Table 2. Corresponding wire sizes and spacings to satisfy these requirements are also presented in Table 2.

In connection with detailing the steel, the following practices are recommended:

1. Depth of steel below surface of concrete should be not less than 2 in. nor more than 1/3 the thickness.
2. Clearance of steel at slab ends and edges should be from 2 in. to 4 in.
3. For welded wire, fabric laps should be to not less than the spacing of the wires. In this case the minimum would be 12 in. for end laps and 6 in. for edge laps.

Pavement slab and wire fabric details are shown in Fig. 2.

This reinforcement design was applied to a 7,300 ft main runway and parallel taxiway constructed in 1956 and in 1959 to additional taxiways, aircraft service

TABLE 2

	Longitudinal			Transverse		
Slab thickness, in Inches	Steel area per foot width, A_s, in square inches	Wire size gage	Spacing, in Inches	Steel area per foot width, A_s, in square inches	Wire size gage	Spacing, in Inches
8	0.142	0	6	.047	2	12
10	0.156	0	6	.052	2	12
12	0.175	00	6	.058	1	12

and warmup aprons for heavy aircraft. The work being designed for 1961 construction will be modified only for depth of slab due to the loadings of the very heavy jet aircraft.

It should be noted that this type wire reinforced pavement is installed with but little, if any, more difficulty than with plain pavement. The reinforcing wire was delivered to the site by truck in flat sheets 12 ft 6 in. by 16 ft which allowed the proper lap and permitted easy handling in the forms.

The concrete was placed in 7 in. and 5 in. lifts by pavers with strike off of the first lift, placement of reinforcement, and the top lift then placed and finished with standard equipment. Two mixers were utilized, a high capacity mixer placing the lower and heavier lift and a smaller paving mixer followed the placement of the wire; thus, the contractors' paving train moved through the job at a steady rate giving high and economical production.

The transverse joints spaced 75 ft apart were sawed within 8 hr to 10 hr after the concrete was placed. Longitudinal joints in the lighter paving sections were sawed and sealed after the 7-day cure period. It is interesting to note that of all transverse joints on the paving work at this airport only six joints in the 1956 work and three in the 1959 work cracked outside the intended saw lines. In each case, the cause was attributed to rapid temperature drops at

night setting up stresses in the slab that caused the slab to crack ahead of the saw, usually after 1/3 to 1/2 of a slab had been sawn. No other cracks have been noted on any of the work to date.

A similar method was used in the design of the reinforced concrete pavement for the critical runway end sections, taxiways, and parts of the aprons at the Bunker Hill Air Force Base, Bunker Hill, Ind. The loadings were heavier and the slab thickness varied accordingly. The sections for this project were

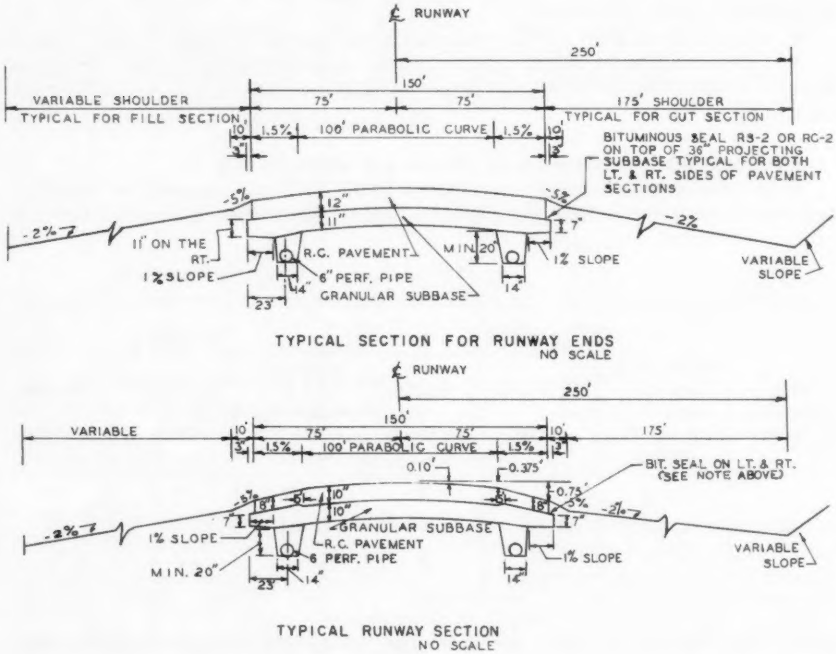

FIG. 2.—TYPICAL SECTIONS AND STEEL DETAILS FOR WEIR COOK AIRPORT, INDIANAPOLIS, IND.

14 in. and 17 in., with transverse joints in the reinforced sections at 75 ft spacing. The unreinforced non-critical sections had joints at 25 ft intervals. This contractor used a battery of three dual pavers, two ahead of the wire placement and one following to obtain continuous and smooth movement of the paving train. The finishing and jointing were routine.

CONCLUSIONS

The use of properly distributed wire reinforcement in portland cement concrete airfield pavements controls cracking; therefore, preserves the smoothness of the slab, reduced maintenance by permitting much longer sections, that is, fewer transverse joints, and under conditions where the slab may be overloaded as is currently happening on many of our existing airports and can also

be expected in the future, maintains the pavement in safe use for a greater span service life than unreinforced pavement. A definite bonus period accrues for the airport authority before it is to construct an overlay or replace the runway or taxiway. Most paving contractors who bid airfield paving projects are also involved in the highway program and are therefore usually well equipped for and experienced in the placing of wire reinforced concrete pavement.

Each client's project, of course, is a particular study in itself but where modern transport aircraft types are involved or anticipated and rigid pavement types are indicated and unless particular circumstances dictate otherwise, the economics of service life, cost of construction, the maintenance cycle, and the "big probable"—the weight and frequency of future aircraft operations—as a rule leads to a recommendation for the use of wire reinforced concrete pavement.

ADDITIONAL REFERENCES

1. "Airfield Pavement," Tech. Publication NAVDOCKS TP-Pw-4, Dept. of the Navy, January 1, 1953.
2. "Engineering and Design, Rigid Airfield Pavements," EM1110-45-303, Corps of Engrs., Dept. of the Army, February, 1958.

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APRON DESIGN FOR LIGHT AIRPLANES

By Kenneth K. Wilde,[1] A.M. ASCE

SYNOPSIS

This paper presents information for the design of parking aprons for single engine and light twin-engine aircraft. Standard dimensions are suggested for laying out tie-downs, and various geometric configurations are analyzed. The operational functions of the apron are then reviewed with respect to their needs and to each other.

INTRODUCTION

In visiting numerous airports, the writer has become aware of the variations existing in the methods used for the tying down of light aircraft. The lack of any uniformity among these methods created an interest in finding if there was one best method. If such a design or method could be found, it would not only provide for easier operations, but benefits would also be derived from more efficient use of the apron or from reductions in the area constructed. It often appeared that only after an apron was constructed, was any thought given as to how it would be used. A typical result was that at one airport you might find a separation distance of 55 ft between rows of parked aircraft and 40 ft at the next. This paper, then, is partially directed toward suggesting separation distances and tie-down configurations for single-engine and light twin-engine airplanes.

Note.—Discussion open until October 1, 1960. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Air Transport Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. AT 1, May, 1960.

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DEFINITIONS

Before proceeding further, it is necessary to define precisely certain terms as they are used herein:

1. Tie-down anchor—The method or equipment used to hold physically an aircraft to the apron.
2. Tie-down point—The location on the aircraft to which the tie-down anchor is attached.
3. Tie-down spacing—The distances laterally and longitudinally between tie-down anchors for one aircraft.
4. Tie-down—The tie-down anchors necessary for the safe holding of one aircraft to the apron. (For the airplanes discussed herein, this is two wing anchors and one tail anchor.)
5. Tie-down configuration—The grouping of tie-downs for several aircraft to permit the efficient and safe operation of aircraft within a minimum of apron area.

TIE-DOWN ANCHORS

There are two general methods for the physical anchoring of the airplanes to the apron. For discussion, they shall be called the individual tie-down method and the cable tie-down method. The first of these uses an individual anchor for each tie-down point. In the second, a long cable anchored at periodic intervals is run the length of the apron, and the connecting cables from the wing tie-down points are attached to it. In both methods an individual tie-down anchor is used for the tail.

Both methods are in general use, and both work well. Advocates of the cable method claim that the cable slack acts as a shock absorber during gusty winds. It also has the advantage of being somewhat more flexible in that the separation distance between the wing tie-down anchors can be varied.

Regardless of the method used, a chain with a snap connector or a rope with a minimum tensil strength of about 700 lb should be used to make the connection between the anchor and tie-down point. Figs. 1 and 3 show designs for constructing anchors for both methods on an asphaltic concrete apron. Fig. 2 shows cable connector details and anchor spacing. For Portland cement concrete aprons, the bars used to form the connecting points are simply imbedded in the slab.

TIE-DOWN SPACING

When the method for anchoring the airplane has been selected, the separation distances between the wing anchors, and between the wing anchors and the tail anchor must be determined. This has been termed tie-down spacing. Tie-down spacing naturally is dependent on the distances between the tie-down points on the aircraft. Tables 1 and 2 list these distances for several popular types of aircraft.

Single-Engine Airplanes.—It would be desirable to select an average wing tie-down width separation which, when the anchor was connected to the tie-down point, would form an angle of about 45° to 60° with the pavement. This avoids a direct pull of the tie-down anchor, thus increasing the total force re-

quired to pull it out. The differences between high and low wing aircraft and the various models make this selection difficult.

An average of the wing-to-wing tie-down point distances in Table 1 yields a value of 17.2 ft. In order for an anchor rope to form a 60° angle with the apron for a high wing airplane, this distance would have to be increased by about 7 ft. The wing tie-down anchor separation thus would be 24 ft. Inasmuch as the single-engine fleet is comprised of approximately 85% high wing aircraft, and that inasmuch as for some of the low wing airplanes this distance is also adequate, 24 ft was adopted for the standard. Distances between 20 ft to 30 ft were observed in use at airports and found to be satisfactory.

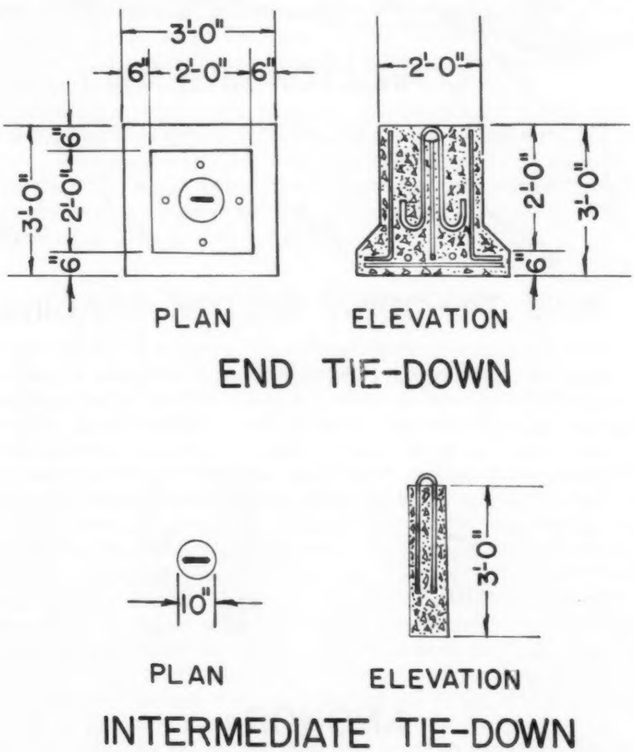

FIG. 1.—TIE-DOWN ANCHORS FOR USE WITH CABLES

The wing-to-tail distance should also be sufficient to form an angle of 45° to 60° between the rope and ground, with the anchor being away from the aircraft. For aircraft with conventional gear configurations where the tie-down point is normally the tail wheel, this angle cannot be obtained. The average for the longitudinal distance between the wing tie-down points and the tail point for single-engine airplanes if 15.0 ft. To this should be added 2 ft to avoid a direct pull on the rope, making a standard length of 17 ft. Once again, distances varying between 15 ft and 20 ft were observed at airports, all of which were satisfactory.

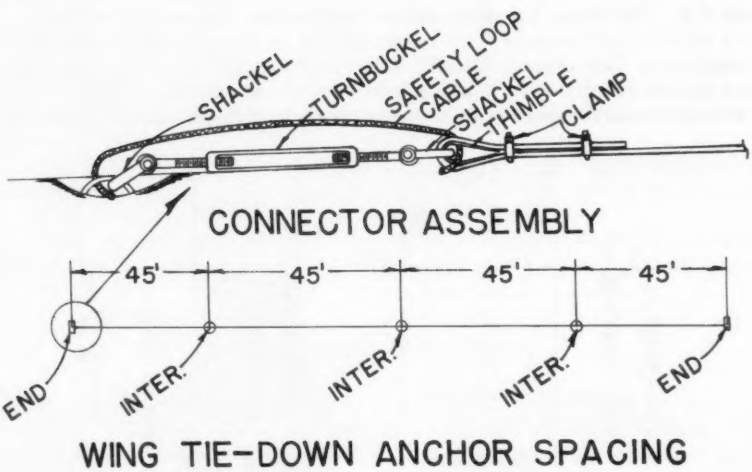

FIG. 2.—CABLE LAYOUT

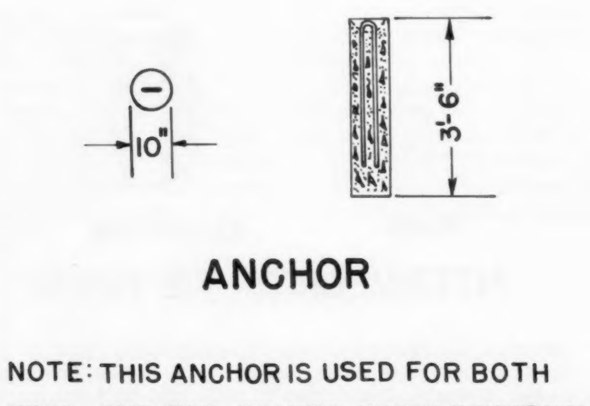

FIG. 3.—TIE-DOWN ANCHORS FOR INDIVIDUAL POINTS

Twin-Engine Airplanes.—A quick review of Table 2 shows that there is little consistency in the wing-to-wing separation distances for the twin-engine airplanes. The average distance is 26.0 ft; however, this is rather meaningless as the range runs from 13.0 ft to 40.5 ft. For certain of these airplanes the single-engine distance of 24 ft would work very well. Since twin-engine airplanes comprise less than 10% of all the active airplanes considered for this study, it therefore appears unnecessary to increase the 24 ft distance for them. The fact that most twins are low winged aircraft is of benefit here. The standard distance of 24 ft, therefore, also would be useable for the light twin-engine airplanes. If there was a sufficient number of twin-engine airplanes at the airport, however, and separate anchors could be justified for them, then a separation of about 32 ft would be in order. Since this distance varies greatly for twin-engine airplanes, it is recommended that a figure be selected most appropriate to the airplanes based at the airport.

There is little problem in the wing-to-tail distance, as it is nearly the same as that for the single-engine airplanes, the average distance being 18.0 ft, only 3 ft greater. This means that for a standard length of 17 ft, nearly a 90° angle will be formed. Although this is not desirable, it is workable as less lifting force is generated by winds at the tail section. For a twin-engine tie-down design, a distance of 21 ft is recommended.

TIE-DOWN CONFIGURATIONS

Before the tie-down anchors can be located on an apron, not only is it necessary to know the required spacing for the anchors for one tie-down of an individual airplane, but it is also necessary to know how the anchors for a number of airplane tie-downs can best be integrated. The object in laying out these tie-downs is to park the maximum number of airplanes in a given area, while still making it easy and safe to taxi and park the airplane. Many configurations are in use. Some of the best of these have been selected so that a comparison could be made of the area required per airplane for a particular configuration.

For laying out the configurations, a standard sized aircraft with a 35 ft wing span and a 25 ft length was used for the designs. This includes nearly all single-engine aircraft and a few of the smaller twin-engine aircraft. With the clearances used, it is also possible to park some of the larger twins. In order to avoid greatly reducing the clearance between aircraft while parking the twins, the airport manager should select two of the smaller single-engine aircraft to be parked adjacent to them. Dimensions of the aircraft in common use are included in Tables 1 and 2.

For certain designs, a comparison was made of the area requirements for single and double lane taxying. Double lane taxying allows for one taxying airplane to pass another. Approximately 30% more area is required for the latter type operation. Double lane taxying is required only when based aircraft number about 200, and even then it should only be adopted if a study of the apron area proves it necessary.

In designing the configurations, the following clearances were used:

1. Between two parked airplanes - 10 ft.
2. Between parked and moving airplanes - 15 ft (single lane taxying).
3. Between parked and moving airplanes - 10 ft (double lane taxying).
4. Between moving airplanes - 10 ft.

TABLE 1.–SINGLE ENGINE LAND AIRCRAFT

Manufacturer and Model	Number of Active Aircraft[a]	Percentage Active of United States Total	Aircraft Dimensions[b]			Distance Between Tie-Down Points[c]		Gear Configuration
			Length, in Feet	Span, in Feet	Height, in Feet	Wing to Wing in Feet	Wing to Tail in Feet	
Aeronca Manufacturing Co.								
11AC, 11BC, 11CC (Chief)	116	1.8	20.4	36.1	7.0	22.0	14.5	Conventional
15AC (Sedan)	262	0.4	25.3	37.5	7.0	27.0	15.0	Conventional
Beech Aircraft Corp.								
35, A,B,C,D,E,F,G,H, & J,K, (Bonanza)	4202	7.0	25.2	32.8	6.5	20.5	16.0	Tricycle
Bellanca Aircraft Corp.								
1413, 14132, 14133	317	0.5	21.3	34.2	6.2	8.0	16.0	Conventional
Cessna Aircraft Co.								
120	1116	1.8	21.0	32.8	6.3	18.0	15.0	Conventional
140, 140A	3054	5.0	21.0	32.8	6.3	18.0	15.0	Conventional
150	100	0.2	21.0	33.3	6.9	13.0	14.0	Tricycle
170, 170A, 170B	3237	5.4	25.0	36.0	6.6	16.0	17.5	Conventional
172	2614	4.3	25.0	36.0	8.5	16.0	16.5	Tricycle
175	597	1.0	25.0	36.0	8.5	16.0	16.5	Tricycle
180, 180A, 180B	1874	3.1	26.0	36.0	7.5	16.0	16.5	Conventional
182, 182A, 182B, 182C	2109	3.5	26.0	36.0	8.5	16.0	16.0	Tricycle
190, 195, 195A, 195B	784	1.3	27.1	36.2	7.2	30.0	18.0	Conventional
Champion Aircraft Corp.								
7AC, 7BCM, 7CCM, 7DC, & 7EC, 7FC, 7GC (Aeronca)	3816	6.3	21.5	35.2	7.0	21.0	14.5	Conventional
Convair Stinson (Universal Aircraft Industries)								
108, 1081, 1082, 1083	2754	4.5	25.2	33.9	7.5	18.0	16.0	Conventional

Forney Aircraft Co.								
E, Fl, G, 415B, 415C, 415D, & 415CD, 415F, (Ercoupe)	2513	4.2	20.8	30.0	5.9	20.0	12.0	Tricycle
Mooney Aircraft Inc.								
M20	173	0.3	23.2	35.0	8.3	9.5[d]	12.0	Tricycle
M20A	110	0.2	23.2	35.0	8.4	9.5[d]	12.0	Tricycle
Piper Aircraft Corp.								
J3C65, J3F65, J3L65, J4A, & J5A (Cub)	4218	7.0	22.4	35.2	6.7	21.0	15.5	Conventional
PA15, PA17, PA18, PA18105SPEC, & PA18125, PA18135, PA18150, & PA18A, PA18A135 (Super Cub)	2064	3.4	22.4	35.3	6.7	--	--	Conventional
PA14, PA16 (Cruiser, Clipper)	582	1.0	23.2	35.5	6.4	15.5	14.0	Conventional
PA20, PA20135, PA22, PA22160, & PA22135, PA22150 (Pacer, Tripacer)	5513	9.1	20.3	29.3	6.2	16.0	13.0	(Conventional (Tricycle
PA24, PA24250 (Commanche)	552	0.9	24.7	36.0	7.4	19.5	13.5	Tricycle
Ryan Navion (Tusco Corp.)								
Navion, A, B, D	1297	2.1	28.7	33.4	8.7	8.5[d]	19.0	Tricycle
Silvare Aircraft Co.								
8 (A to F) (Luscombe)	2553	4.2	20.0	35.0	6.3	20.5	13.0	Conventional
Taylorcraft, Inc.								
BC12D, BC1265, BC65, BL65, & DC065, BC12DI	2325	3.9	22.0	36.0	6.5	21.5	15.5	Conventional
Universal Aircraft Industries								
GC1A, GC1B (Swift)	690	1.0	20.9	29.3	5.9	10.0[d]	13.5	Conventional
Total & Average this Table	50542	83.6%	23.3	34.4	7.1	17.2	15.0	
United States Total	60490	100%						

[a] As of Jan. 1, 1958, source of information, Statistical Study of U.S. Civil Aircraft. [b] Dimensions may vary between models and years. [c] Dimensions are to nearest ½ ft. [d] Distance between main gear.

TABLE 2.—TWIN-ENGINE LAND AIRCRAFT

Manufacturer and Model	Number of Active Aircraft[a]	Percentage Active of United States Total	Aircraft Dimensions[b]			Distance Between Tie-Down Points[c]		Gear Configuration
			Length, in Feet	Span, in Feet	Height, in Feet	Wing to Wing in Feet	Wing to Tail in Feet	
Aero Design & Engineering Co.								
520	108	1.5	36.0	44.0	14.0	--	--	Tricycle
560, 560A,	131	1.9	35.4	44.1	14.75	13.0[d]	19.0	Tricycle
560E	32	0.5	35.4	49.0	14.4	--	--	Tricycle
680, 680E	202	2.9	35.4	44.0	14.5	13.0[d]	19.0	Tricycle
Beech Aircraft Corp.								
AT11	109	1.5	34.2	47.7	--	40.5	19.0	Conventional
(Super 18) 18A, 18D, 18S B18S, D, D, E, (Twin Bonanza)	933	13.2	34.0	47.6	9.2	40.5	20.0	Conventional
50, B50, C50, D50, E50, & F50, G50 (Travel Air)	453	6.4	31.5	45.3	11.3	33.0	18.5	Tricycle
95	168	2.4	25.3	37.8	9.5	20.5	16.0	Tricycle
Cessna Aircraft Co.								
310, 310A, 310B, 310C	629	8.9	27.1	36.1	10.4	26.0	16.0	Tricycle
Piper Aircraft Corp.								
PA23, PA23160 (Apache)	1190	16.8	27.1	37.0	9.5	21.5	17.0	Tricycle
Temco Aircraft Corp.								
D16, D16A (Twin Navion)	74	1.0	27.2	34.0	10.3	--	--	Tricycle
Total & Average this Table	4029	57.0%	31.4	42.0	11.8	26.0	18.0	
United States Total	7073	100%						

[a] As of Jan. 1, 1959, source of information, Statistical Study of U.S. Civil Aircraft. [b] Dimensions may vary between models and years. [c] Dimensions are to nearest ½ ft. [d] Distance between main gear.

It was felt that there was some justification in reducing the clearance between parked and moving airplanes for double lane taxying, since the times that this will occur are limited, and the taxying speeds will be very slow. Based on the above clearances, an isle width of 65 ft is required for single lanes and 100 ft for double lanes.

Adoption for a configuration will depend on the following conditions:

a. Number of airplanes.
b. Direction and velocity of winds.
c. Area to be developed.

Figs. 4 through 10 show several apron designs patterned after designs in actual use at airports. All of the designs are usable. For larger airports where area is scarce it may be necessary to park the airplanes closer together, thus requiring a design as shown in Figs. 5 or 7. When space is not at such a great premium, a design like Fig. 6 can be used, eliminating any moving of airplanes by hand. This same design would be good in locations where

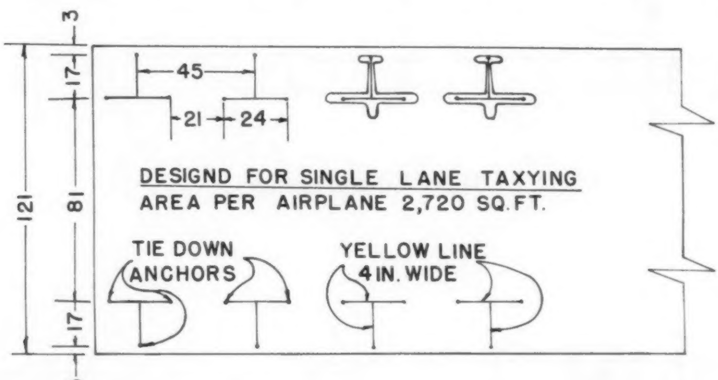

FIG. 4.—APRON DESIGN

high winds dictate that all aircraft be faced into the prevailing direction. When the area available for development is narrow or oddly shaped, it may be necessary to adopt a design that will best fit it.

It will be noted in Fig. 4 that a 4-in. wide yellow line has been shown connecting the tie-down anchors. Painting this line was found quite useful where it was in operation. It clearly shows the pilot where he should position his airplane and eliminates any mix-up in using the anchors by clearly defining each tie-down position.

APRON SIZE AND LAYOUT

With the size and configuration of the airplane tie-downs established, the greatest remaining problem confronting the designer is, "How large should the apron be?" This, of course, is dependent on the activity at the airport. A logical approach to figuring this size is to analyze the individual needs of the parking apron.

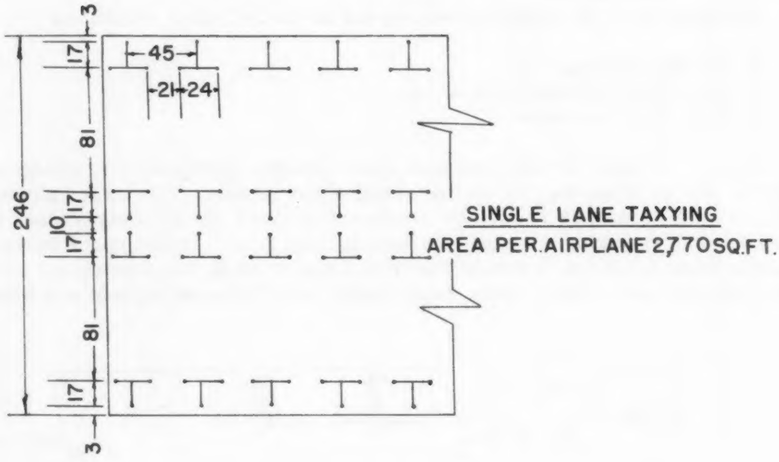

FIG. 5.—APRON DESIGN

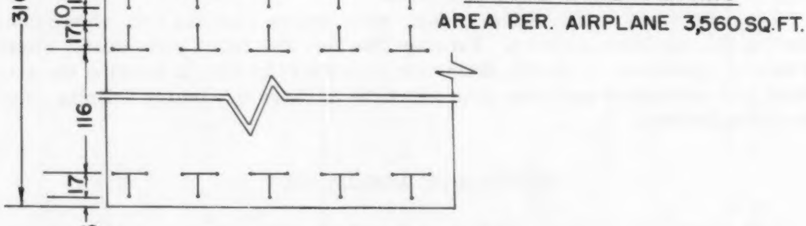

FIG. 6.—APRON DESIGN

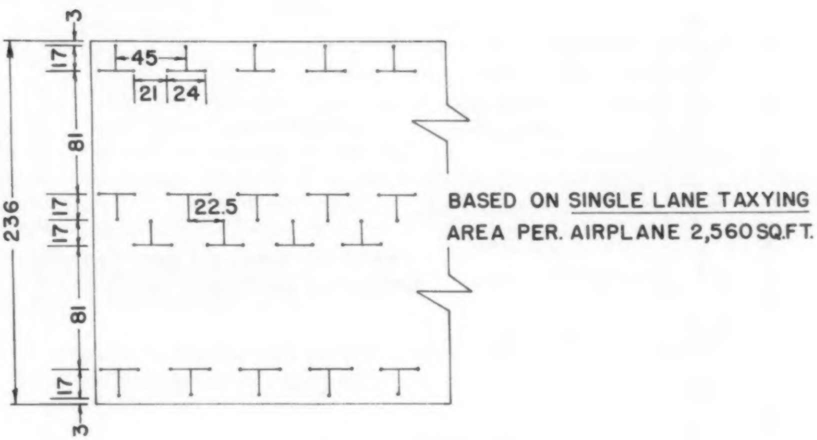

FIG. 7.—APRON DESIGN

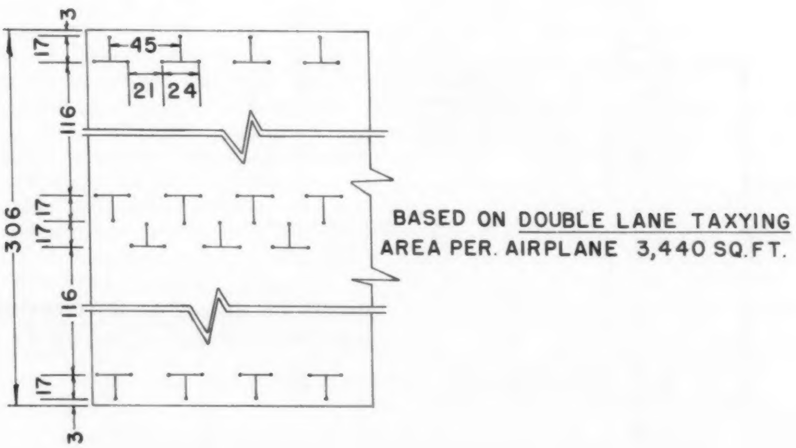

FIG. 8.—APRON DESIGN

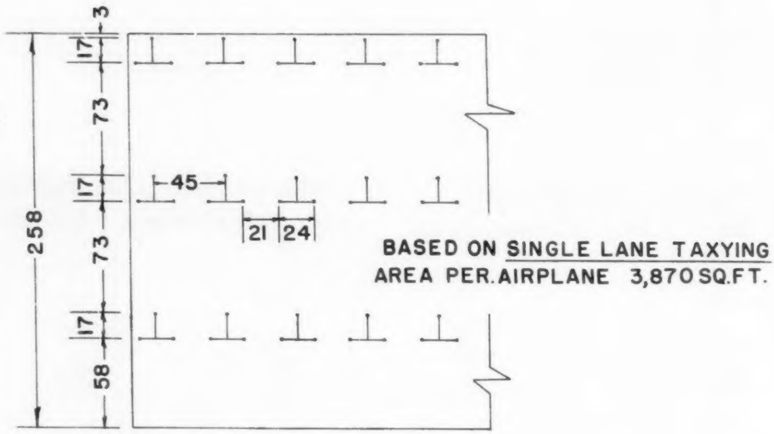

FIG. 9.—APRON DESIGN

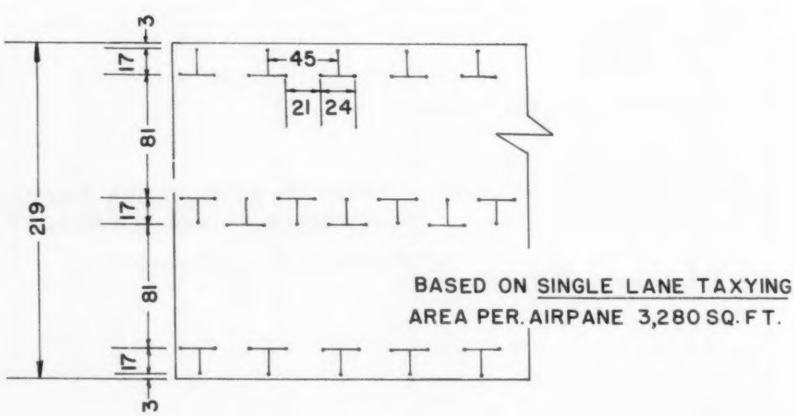

FIG. 10.—APRON DESIGN

The total apron area may be divided into the following five operational functions:

1. Based airplane apron.
2. Transient airplane apron.
3. Fixed base operator apron.
4. Fueling apron.
5. Unloading apron.

A knowledge of these functions is paramount to the successful design of an apron, as well as an understanding of the relationship of one function to another. In a well-planned apron the various functions should fit together like pieces of a puzzle, working in harmony for all its users to produce a smooth operation. The role that each function plays in the operation of the airport must also be considered then, and along with this the needs of the people involved.

The following discussion will be aimed at answering these three questions for each of the five functions:

a. What is its function and needs?
b. How big an apron is required?
c. Where should it be located?

Each of the five will be discussed individually.

TRANSIENT APRON

A certain portion of any apron must be devoted to transient airplanes. A transient pilot may be just stopping for fuel and food, for several hours, or over-night. Whatever his purpose, he should be extended the best service possible. This includes among others, a choice tie-down location on a paved apron with easy access to rest rooms, terminal building, and fuel. The transient pilot probably will not be familiar with the operations of a particular airport. It is pertinent, therefore, that he be allowed to park near the terminal, a natural destination, and not sent to an out-of-the-way spot.

As this type of apron will often be vacant, it is necessary to keep its size to a minimum. However, sufficient area should be set aside to accommodate an average maximum number of transient airplanes. During "Fly-ins" or other events attracting increased traffic, special areas for transient parking, often on grass, can be set up along with special signs directing the pilots to it.

The average maximum number of airplanes is the greatest concern. This will vary with the particular airport. One with an outstanding restaurant or in a resort area will have a higher number of itinerant aircraft than otherwise. If the airport is already in existence, some ratio of itinerant to based airplanes can readily be established. For new airports a figure of about 10% of the based airplanes can be used. This should be increased by 5% to 10% if located in a resort area or if the airport has some other distinctive attraction.

BASED AIRCRAFT AND FIXED BASE OPERATOR'S APRONS

The majority of the apron area will be devoted for the use of the airplanes based at the airport and for the fixed base operator. These functions are dis-

cussed together, for the apron areas are often used jointly. The fixed base operator's airplanes are normally counted and tied down along with those of private owners. He may even rent his tie-down space by the month based on actual need, rather than pay a flat rental for use of the apron.

For these two functions, the location need not receive as high a priority with respect to the terminal building and other facilities. The pilots know the airport, where they are going, and except for fuel, may not need other facilities. At larger airports, it may even be necessary to locate part of the based aircraft apron at a spot remote from the terminal area. It is necessary, however, to have adjacent automobile parking facilities, so that all such traffic can be kept clear of the operational areas.

In addition to his needs for storage of airplanes, the fixed base operator will require a certain amount of apron adjacent to his hangar if he offers repairs or other service. This will be a clear area, probably without tie-down anchors, used only for the intermittent parking of airplanes while servicing them and for entrance to his hangars. Anticipating the needs of the operator is difficult, as it depends on the business he generates. It is sometimes best to lease him land adjacent to the apron, and to allow him to construct his own apron. This shifts the burden of deciding how much apron to build from the airport owner, and also relieves him of the construction costs. The airport owner, however, should retain the right to direct the operator as to where he will build the apron. No matter who owns or uses an apron it must have continuity with the airport Master Plan.

For planning the size of the combined aprons the number of based airplanes (including the fixed base operator's) should be anticipated for the next one to two years. A certain percentage of the owners of these airplanes will desire hangars, if these are available. In the most favorable of climates this percentage will be in the order of 10% to 20% of the based airplanes. It will naturally increase for more severe climates, such as ones with high winds or heavy snows. Another factor affecting this percentage is the value of the airplanes. For newer airplanes or twin-engine airplanes the cost of a hangar rental is not too great a proportion of the overall operating expense.

FUELING APRON

If a service station for refueling airplanes is to be used, then an apron area for this purpose must be planned. The choice of a service station or truck system operation is often one of personal preference and must be decided prior to the design of the apron system. Stations nearly always are used on airports of low and moderate activity.

A service station is most efficient if it is able to service airplanes on all sides or a 360° servicing area. This allows for a maximum number of airplanes to park near it with a minimum of moving of airplanes by hand, if any. With this design, up to five airplanes can be parked within range of the pumps, and require no hand moving. On airports of low activity, however, this design will take too great a percentage of the area, and also is not required. In this case, a pit adjacent to the apron and capable of fueling two airplanes is adequate. Fig. 11 shows the above two designs with the area required for the parking of the adopted standard 35 ft by 25 ft airplane.

The fueling apron should be located near the terminal building and transient apron. Attendants who may have other duties in the terminal can then watch the

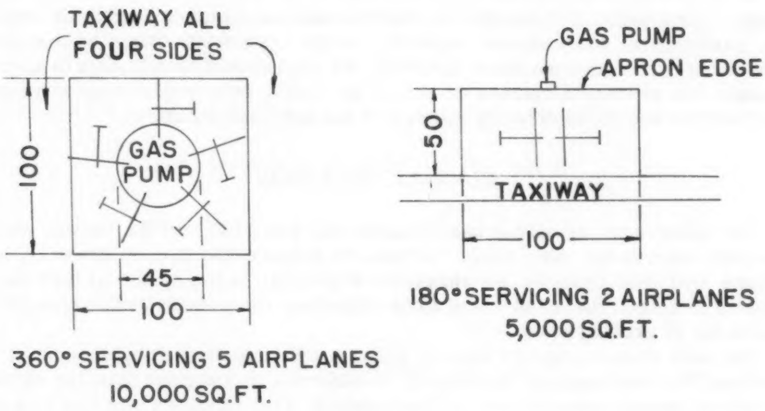

FIG. 11.—FUELING APRON

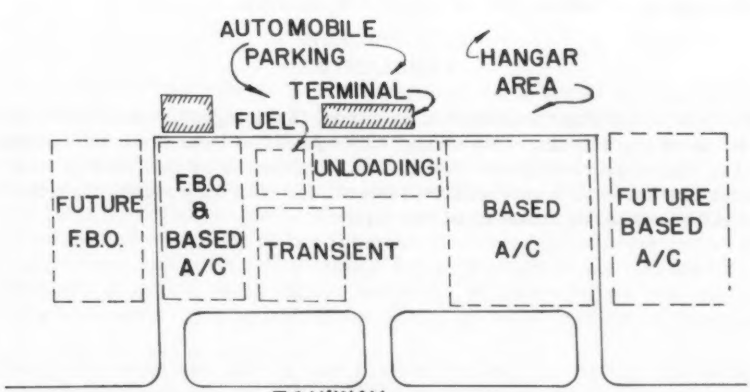

FIG. 12.—PRELIMINARY APRON PLAN

station and quickly give service when needed. With this location it is easy also for the attendant to park by hand any airplane left at the station for fueling and for tying down.

UNLOADING APRON

At larger general aviation airports, there is often a need for an unloading apron. This apron will be used by charter aircraft for discharging or enplaning passengers, for business aircraft, or for anyone who wants to stop for a few minutes to complete some business. Its size should be adequate to accommodate two average airplanes or one of the larger twin-engine type airplanes. Its location should be directly in front of the terminal building.

PLANNING AN APRON

The apron area usually is best located near the middle of the runway and on the side nearest the main road. Terrain or airport boundaries, however, may dictate that this practice be changed. When this is the case, all that can be done is to select the most level area affording the greatest expansion with a minimum of grading costs.

The size of the complete airport apron will be determined by the sum of the needs of the five functions previously considered. In designing this, the various functional aprons should first be interrelated. Fig. 12 shows one way in which this can be done. Each airport will require its own analysis. Approximations as to the overall size are sufficient to start with. After a basic plan has been established, taxiways can be added, tie-down configurations adopted, and exact dimensions given to the areas to accommodate the required aircraft.

Provision should always be made in the beginning for additional growth of the apron. It is usually unwise, therefore, to place buildings at the apron ends, as this is often the only direction for expansion. The tie-down configuration should also be chosen with expansion in mind. If not, it may be later necessary to waste area or to remove and to install new anchors.

CONCLUSIONS

Prior to beginning the design of an apron, the designer should first analyze the needs of the airport. This should encompass the type, size, and number of aircraft that it must support. From this information he will be able to select standards which will insure efficient operation. The only remaining act, then, is to tailor-make the apron to fit the airport.

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AIR TRANSPORT DIVISION
Proceedings of the American Society of Civil Engineers

AERIAL PHOTOGRAPHY IN ARCTIC AND SUBARCTIC ENGINEERING

By Robert E. Frost,[1] A. M. ASCE

SYNOPSIS

The analytical airphoto procedures developed in temperate climates are equally applicable to engineering problems in cold regions. Included are: (1) consideration of the regional environmental aspects of the area; (2) detailed study of the minute characteristics and configurations composing the pattern; and (3) recognition and evaluation of the surface configurations which are the result, specifically, of permafrost and/or severe frost activity. The paper illustrates the foregoing and reviews the present "state of the art."

INTRODUCTION

The purpose of this paper is to discuss the use of aerial photographs as a means of getting information about surface materials and their condition which will assist in solving problems of scientific, engineering, or military nature in arctic, subarctic, and polar areas. This discussion is limited to: (a) a short review of permafrost and severe frost activity as an arctic - subarctic problem; (b) a brief review of the airphoto method of obtaining information; (c) determination and evaluation of the regional environmental aspects responsible for permafrost and severe frost action and related problems; (d) identity and significance of minute photo pattern features; (e) direct permafrost and frost activity indicators; (f) a typical analysis; and (g) a few concluding statements.

Note.—Discussion open until October 1, 1960. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Air Transport Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. AT 1, May, 1960.

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PERMAFROST, SEVERE FROST ACTIVITY AND THE PROBLEM

Permafrost is a condition of earth materials, including bedrocks, which exists at temperatures continuously below freezing in a generally solid state. The areal extent of permanently frozen soils is quite large. About one fifth of the dry-land surface of the earth is underlain by permanently frozen soils. The results of deep drilling programs have shown that the permafrost condition varies in thickness from a few feet in milder portions of northern lands to over 1,000 ft in favored locations, in extreme northern areas. On a regional basis, the presence of permafrost in an area is largely dependent on climate. On a local basis the type of permafrost occurring is largely a function of the natural and physical local environmental features. This includes topographic position, landform, slope, exposure, vegetative cover, surface drainage, and soil-parent materials. Evaluation of these features, whether accomplished by ground inspection or by the use of airphotos, is of paramount importance in planning any activity which will disturb the surface features in arctic and subarctic regions.

With regard to the effect of permafrost on engineering design, location, and construction it has been found that both good and bad soil situations occur in permafrost regions. The importance and severity of permafrost as a problem condition is largely a function of the ice-soil relationship in that portion of the soil/rock mantle having a temperature below freezing. For the most part the good construction areas contain well-drained granular materials which occur in elevated positions in comparison to the surrounding terrain. Coarse-grained soils (boulders, gravels, coarse sands) having little or no moisture content may exist in a loose, friable state even though the temperature may be below freezing (Figs. 1 and 2). In depressed or poorly drained positions, such soils may be saturated and may exist in a solid state because of the frozen interstitial water. In either event such soils can withstand thaw and disturbance without causing any appreciable difficulty.

The most adverse permafrost condition is that related to fine-textured soils containing a large percentage of ice in the form of crystals, lenses, wedges, sills, or massive accumulations of ice (Figs. 3 to 6). When such are situated low topographically or on extremely flat areas, the permafrost problem becomes extremely critical. Identification of this latter condition is important because thawing of this type of frozen soils is accompanied by severe volume change and loss of supporting power. Often the structures will experience severe distress almost immediately following construction (Figs. 7 and 8). Thaw is usually continuous under heated structures which have not been insulated properly from the ground surface. The destruction, or merely disturbance, of the protective natural insulation renders such frozen areas non-useable for structures and non-trafficable for most vehicles (Figs. 9 and 10). Recovery of permafrost of this type in disturbed areas is extremely slow, even after completely abandoning a disturbed site.

The permanently frozen soil mass is overlain by an "active layer" which freezes and thaws every year. The active layer is critical from the standpoint of movement of soils contained therein due to frost activity, gravity, solifluction, and other forms of earth-movement processes. In areas where a high percentage of silt occurs in the active zone, consideration of the active zone becomes more important than considerations of the permafrost zone for certain types of structures, particularly those structures which cannot withstand seasonal movement for successful performance. It is important that the properties of the active zone be determined so that disturbance of the often delicate

FIG. 1.—PORTIONS OF THIS GRANULAR TERRACE ARE WELL DRAINED AND UNFROZEN.

FIG. 2.—HIGHWAY CUT IN THE OUTWARD PORTION OF A DRY UNFROZEN GRANULAR TERRACE

FIG. 3.—TEST PIT ACROSS A POLYGON EXPOSING ICE WEDGES WHICH OCCUR BENEATH POLYGON CHANNELS.

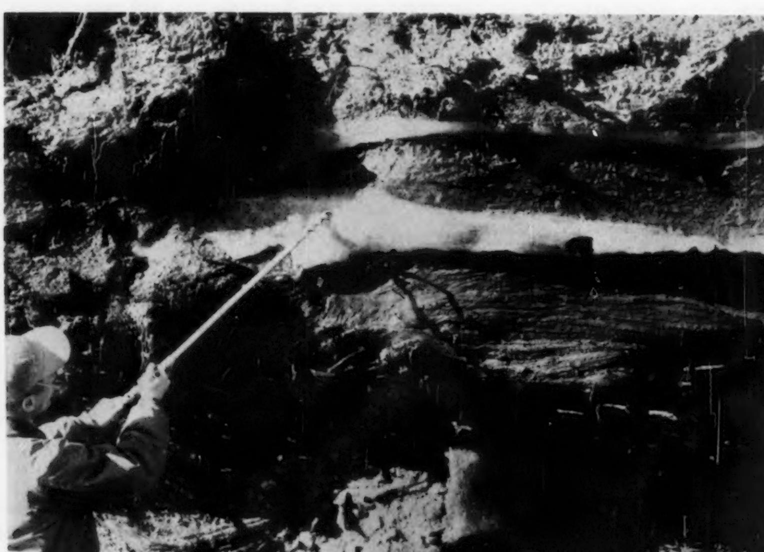

FIG. 4.—LEDGES OF ICE IN SILL FORM IN VALLEY FILL DEPOSITS.

FIG. 5.—ICE CRYSTALS IN FINE GRAINED SOILS

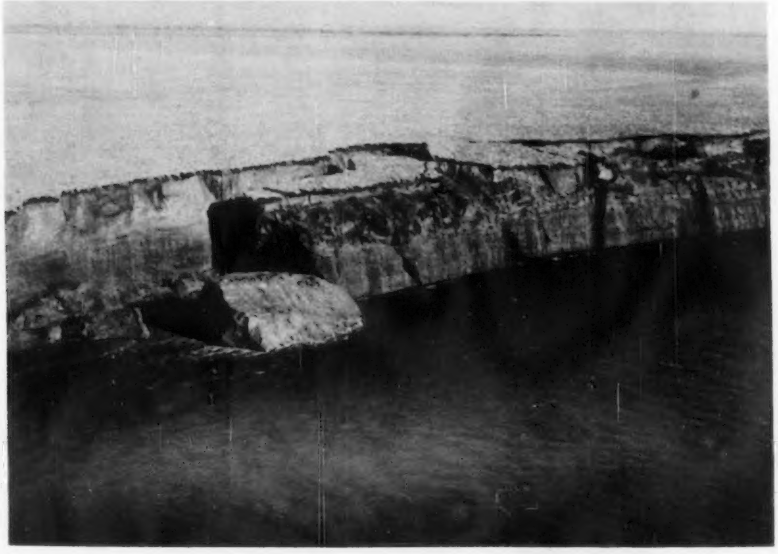

FIG. 6.—MASSIVE GROUND ICE ON FROZEN SAND AND SILT BEING UNDER CUT BY THAW ACTION OF THE WATER.

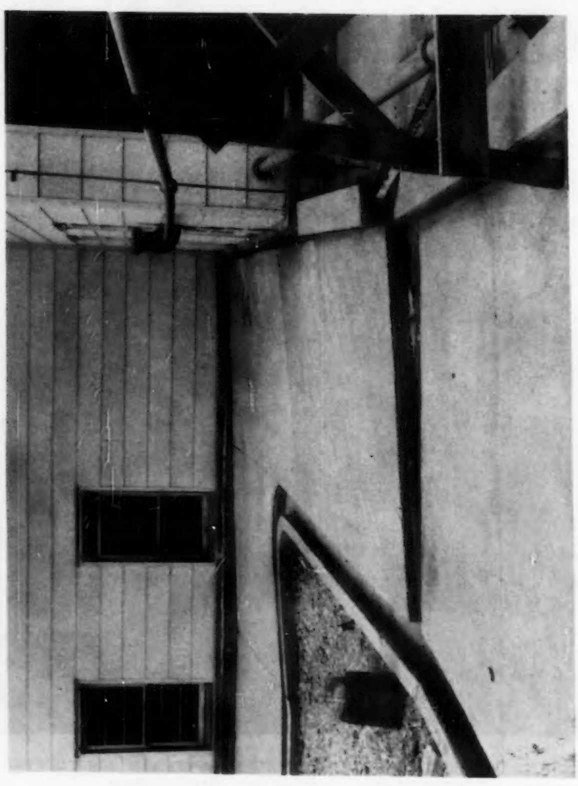

FIG. 7.—THE EXTENT OF DIFFERENTIAL MOVEMENT BETWEEN THE BUILDING AND ITS SURROUNDINGS INDICATES THE SENSITIVITY OF THE PERMAFROST THAW PROBLEM.

FIG. 8.—AN EXAMPLE OF VERY SEVERE SETTLEMENT IN A BUILDING RESULTING FROM THAW.

FIG. 9.—ONE OR TWO PASSES WILL DESTORY THE INSULATION AND RESULT IN A SEVERE THAW.

FIG. 10.—CONTINUED TRACKING OF VEHICLES WILL SOON RESULT IN A SERIES OF CANALS.

thermal regime can be minimized. In an undisturbed state the active layer, together with its protective cover, plays an important part in preserving the permanently frozen layer by protecting it from the summer heat.

The engineering problems are contingent on the presence or absence of permafrost, which type exists, susceptibility of the active zone to severe frost activity, and where the structure is with respect to the permafrost-active zone profile. Structures confined to the active zone are subjected to seasonal freezing and thawing which will be accompanied by considerable movement if the soils are finely textured. This is particularly objectionable for such structures as highways, railroads, power lines, buildings, pipe lines, and utilidors. The period of most severe damage is during and shortly following the spring breakup. It is during this period that paved and unpaved road and airfield surfaces suffer the greatest distress (Fig. 11). If situated high topographically, such areas may recover rapidly once the frost leaves and the water drains away. If situated in depressed or flat topographic positions such soils may never recover.

Structures placed on frozen materials having low moisture content do not usually experience serious damage from the resulting thaw. Airfields, runways, roads, and buildings have been placed on frozen sands and gravels and have performed satisfactorily not only following construction but throughout the critical breakup seasons as well (Fig. 12). There are, however, often local areas containing much ice which lies within the bounds of dry frozen materials which should be avoided in engineering construction.

Identification, analysis, and evaluation of the soils and permafrost occurring in an area or deposit in the arctic, subarctic, and polar regions being considered for engineering use becomes a critical matter. Often time does not permit conducting detailed examination of the natural and physical environmental features of an area by field procedures. The far northern latitudes experience a very short summer field season. Many areas which offer good potential for the location of large structures are almost completely inaccessible to present day transportation means (1960). The strategic importance often creates a great urgency for obtaining information about remote areas which can be used in all problems concerned with earth-surface features. It is believed that the present "state of the art," or present state of development of airphoto analysis and interpretation, fulfills the majority of these requirements.

AIRPHOTO METHOD OF OBTAINING INFORMATION

The more reliable methods of obtaining information are based on research which was designed to coordinate aerial imagery with ground sampling in order that the many variables affecting information content in a picture could be defined and evaluated. Aerial images utilizing a broad range of the visible portion of the spectrum had to be tested for their faithfulness of representation of ground features within the wave length spread utilized by various films and filters and the system used. This research has proven fruitful from the standpoint of conventional optical photography. Interesting and significant trends are being established for sensor systems which seek information far beyond the visible portion of the spectrum.

The use and application of aerial photographs in arctic and subarctic areas follows the same general analytical procedures which were developed in and which are applicable to temperate climates. The completely analytical method

FIG. 11.—SEVERE ROAD FAILURE RESULTING FROM THAW OF FROST HEAVE IN SILTY SOILS.

FIG. 12.—UNPAVED AIR STRIP ON DRY FROZEN SAND IN THE ARCTIC COASTAL PLAIN.

of using airphotos for obtaining information about the soil mantle and its condition (frozen, frost acting, etc.) consists of applying logic and deductive reasoning to: (1) consideration of the regional environmental aspects of the area, (2) detailed study of the minute characteristics and configurations composing the pattern; and (3) recognition and evaluation of the surface configurations which are the result specifically of permafrost and/or severe frost activity.

REGIONAL ENVIRONMENTAL ASPECTS

Regional consideration includes geography, physiography, geology, and climate. The geographic aspects include physical and political location, size and shape of the area, boundary conditions, proximity to major land masses or water bodies, and major interest or cultural pursuit. Much of this, by necessity, must come from other sources.

The study of physiography is concerned with the relationships which exist between the major land units or land masses on a regional basis. The present landscape is an expression of all of the forces which have contributed to its development. These forces are related to deposition, erosion, orogenic movement, or any combination of the three. Consideration of the physiography of an area provides an important clue about the materials and their physical characteristics on a large scale. The arctic - subarctic landscape is altered to varying degrees by certain of the processes related to permafrost and severe frost activity. The most severe alteration occurs on land masses having a fine-grained soil and/or soil-rock mantle in which the confined forces of freeze-thaw and gravity result in severe surface movement. In such instances, the arctic landscape appears considerably rounded—slopes are soft and marks of movement occur everywhere. Coarse textured soil mantle and/or resistant rocks create the most rugged forms and slopes. In sloping areas the upper layer or top of the permafrost often acts as a good slipping zone or plane over which the materials in the active zone can move.

From the standpoint of geology, the analyst is interested in origin of deposits and the sequence of events responsible for development of the landscape. Each major parent material, whether developed in place and considered residual or transported by the action of wind, water, ice and/or gravity, has its own identifying pattern characteristics. Determination of the geology limits the type of materials expected to be found as well as degree and type of permafrost - frost activity in an area.

If meteorological data are not available the general climatic condition can be obtained from the airphotos but by inference only through the study of vegetation (its type and distribution) and by study of the erosional features. Knowing the general climatic condition under which the area has existed and under which the soils have formed from a given parent material gives an additional clue to the type of soil, the profile development, and, to a limited extent, permafrost and the susceptibility of seasonal frost activity. If the regional analysis has been successful, then the analyst knows something of location, size, and major area interest; has learned much about the physical expression of the major topographic forms; knows how the deposits were formed and what processes were active in shaping the area and what the major parent materials are, and; from the climatic indications, he has a good idea of presence or absence of permafrost and/or the severe seasonal frost potential. The regional study results in dividing an area into major patterns which are usually coinci-

dent with landform - parent material types. The landform, parent material types provide a basis for understanding what to expect with respect to problems related to location, design, construction, and, possibly, maintenance of large structures.

PHOTO PATTERN FEATURES

Each earth material is identified by the pattern features or elements, which it reflects. The common elements include landform, drainage, erosion, vegetation, tones, cultural features, and any unique or special features. It is to be stressed that detailed stereoscopic study of pattern elements results in determination of physical characteristics of materials and their state or condition. The presence of permafrost and/or severe frost activity does exert considerable influence on the appearance of some of the pattern elements of fine-grained soil and/or some rock types.

Analysis of the landform results in obtaining information about the composition of a deposit both areally and vertically. Landforms related to coarse-grained materials are little altered by the presence of permafrost while those which contain fine-grained soils may be altered considerably (Figs. 13 and 14). The surface characteristics will be altered considerably by severe frost activity if the active zone contains fine-grained soils and a source of moisture. Topographic position is important in determining the type and degree of permafrost - frost activity. The high or upland situations associated with coarse-grained materials offer no particular problem. The upland areas which contain fine-grained soils in depressed position can be expected to offer serious difficulties. The most severe topographic positions are related to the transition zones between high and low areas where the low areas are receiving fine-grained mantle from higher positions. Slope and exposure are items which must be studied in great detail since they greatly influence permafrost and thickness of the active layer. In general, the active zone is much thicker on south-facing slopes than on the north-facing slopes. This is of considerable importance from the standpoint of workability of engineering materials situated on sloping ground.

The drainage pattern reflects such things as topographic arrangement, general porosity, relative depth of soil mantle (if present), type and depth of rock (if present), and the areal extent and structure of either mantle or rock beneath the mantle. The regional drainage plan or pattern is little altered by the presence of permafrost. Locally, the drainage plan is altered considerably due to the presence of permafrost. The active zone and the frost-activity problem does not significantly affect the major drainage networks.

Erosional aspects provide an important clue to composition, condition, and uniformity of a deposit. In warm environments such characteristics as the gully arrangement, cross section, and profile are related to soil texture, soil structure, profile development, slope, and state of ground. In general, changes in any of the reflected characteristics means changes in materials or condition. Permafrost alters the erosional aspects of fine textured soils completely.

For the most part frozen soils erode by thaw which can be induced either naturally or through activities of man. A sudden increase in surface water in hilly areas may result in severe damage or possible destruction of established polygons (Figs. 15 and 16). The channels of the polygons become gullies and

FIG. 13.—THE CHARACTERISTICS OF THE LANDFORM OF A WELL DRAINED GRANULAR TERRACE ARE LITTLE ALTERED BY THE PRESENCE OF THE PERMAFROST CONDITION.

FIG. 14.—THE SURFACE CHARACTERISTICS OF PERMANENTLY FROZEN SILT COVERED TERRACES ARE ALTERED CONSIDERABLY BY THE PERMAFROST CONDITION

FIG. 15.—DESTRUCTION OF POLYGONS BY THAW.

FIG. 16.—CONTINUED THAW OF POLYGONS RESULTS IN A SERIES OF SILT MOUNDS.

destruction of the net by thaw and removal is often rapid. The ice in the channel thaws first causing a visible subsidence in the vegetation mat (Fig. 17). If thaw occurs on the surface of a polygon area then a drainage pattern of geometric regularity is exhibited—this is also easily recognized on airphotos (Fig. 18). Continued thaw and removal may result in leaving only isolated soil mounds as remnants.

Another common erosional feature in permafrost areas is the occurrence of "caving blocks" of frozen soils along the banks of streams and shores of large bodies of water (Figs. 19 and 20). The warm water of a stream, or lapping waves of the ocean, tends to induce waterline thaw which results in breaking off large blocks of frozen material. Thermokarst lakes provide another form of thermal upset in permafrost areas which results in a recognizable pattern on airphotos (Figs. 21 and 22). Thermokarst lakes are depressed basins in permafrost areas which are believed to have been formed by thawing of large masses of ground ice in an unstable permafrost regime. The lakes often are characterized by a scalloped or serrated shore line, and when occurring in timbered areas they are fringed by leaning trees (Fig. 23). In good quality airphotos it is possible to see overhanging mats of vegetation along the shore line of such lakes—another indication of thaw. One form of thermokarst pattern accompanies the sursurface thaw of polygons. The thawing action is influenced by ground slope, exposure, vegetative cover, soil texture, and surface water. If a polygon area has been upset by subsurface thaw, perhaps as a result of the destruction of the native cover, considerable subsidence occurs which results in a hummocky area having a pattern which is easily recognized on airphotos.

One of the most common types of gully systems in permafrost regions is that associated with "button drainage." Button gullies are usually found in association with polygon areas which are in the initial stages of thaw. Thawing usually starts at the intersection of polygon channels and small circular pools of water result. Usually flow is hardly perceptible. A chain of connected buttons forms the pattern (Fig. 23). The placer-type gully is another erosional feature common to permafrost areas and usually reflects undesirable permafrost conditions. Gullies of this type exist as broad softly rounded troughs which do not contain a definite channel. As long as such features are undisturbed, an upset by thaw will not result. Great quantities of water are stored in the vegetative mat, which retards the flow and prevents erosion.

Vegetation is an important pattern element as certain species are used as indicator species. The vegetation in an area provides a single expression into which are integrated all of the significant physical, chemical, and biological factors brought to bear upon it. Vegetative cover is of importance in arctic regions because of the protection afforded to the underlying permafrost and provides clues to type and degree of permafrost and conditions in the active zone. In timbered areas white spruce-paper birch forests occur on both frozen and unfrozen soils; black spruce-tamarack stands grow on frozen muskegs; aspen occurs on dry unfrozen, south-facing slopes; balsam-poplar stands are confined to sites adjacent to active streams having moist, sandy soils unfrozen to a depth of at least 10 ft or more; pure, dense, willow stands grow on bare river bars which are unfrozen to 10 ft or more; and pure alder brush occur on wet peaty soils frozen at a depth of 30 in. In tundra areas the cover includes niggerheads, mosses, sedges, grasses, lichens, berries, varieties of tea, dwarf birch, dwarf willow, low brush, and cotton sedges. Trees and large bushes are not found in tundra areas. Alders grow on slopes of hills and in major gullies, chiefly where protection from severe winds is afforded. Some willows,

FIG. 17.—AS THE ICE IN A POLYGON CHANNEL THAWS THE OVERHANGING MAT OF VEGETATION IS LEFT UNSUPPORTED AND CRACKS BEGIN TO DEVELOP.

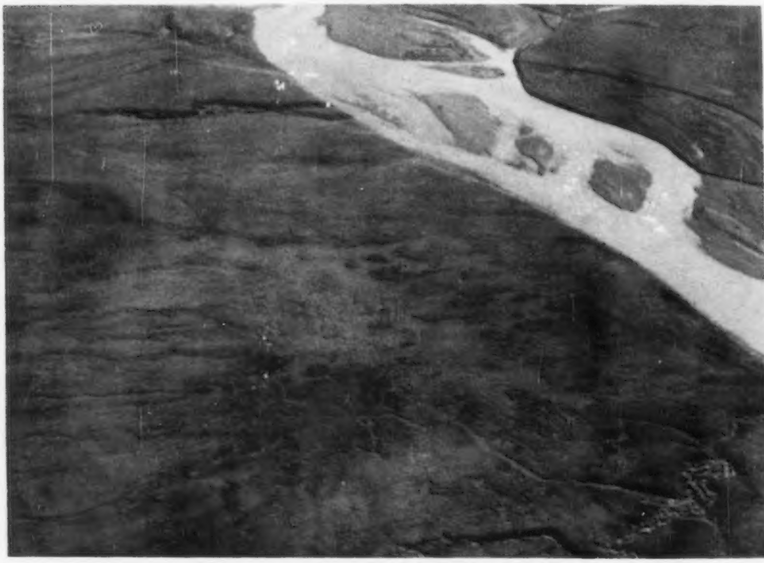

FIG. 18.—FREQUENTLY GULLIES EXHIBITING A GEOMETRIC DESIGN OCCUR IN POLYGON AREAS.

FIG. 19.—UNDERCUTTING OF STREAM BANKS BY THAW LEAVES LARGE BLOCKS OF FROZEN SOIL UNSUPPORTED.

FIG. 20.—LARGE BLOCKS OF FROZEN SOIL WHICH HAVE BROKEN OFF FROM THE STREAM BANK.

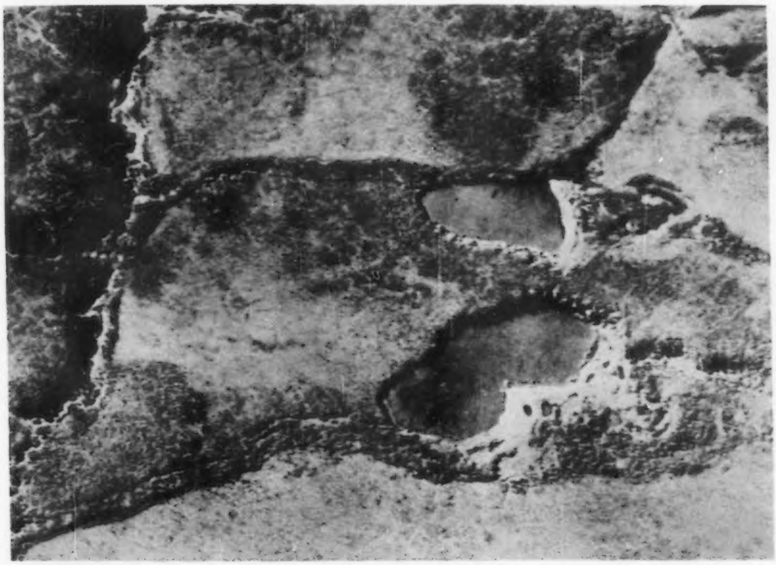

FIG. 21.—TWO THERMOKARST LAKES WHICH HAVE RESULTED FROM THAW OF MASSES OF GROUND ICE.

FIG. 22.—SMALL THERMOKARST LAKES.

FIG. 23.—THAWING STREAM BANKS ARE OFTEN CHARACTERIZED BY LEANING ("DRUNKEN") TREES.

FIG. 24.—BUTTON DRAINAGE IS ONE OF THE OUTSTANDING INDICATORS OF PERMANENTLY FROZEN SOILS.

often 12 ft to 14 ft high, occur along the water courses in parts of the Alaskan Arctic Coastal Region.

The element photo tones provides an additional clue concerning the materials and their condition. For the most part, changes in tones reflect changes in materials and/or their condition. The tone differences must be evaluated with respect to topographic changes, vegetative changes, and/or land-use changes before their true significance can be determined. The presence of permafrost may result in an appreciable darkening of tones because of the often higher moisture content of the active zone. Depressed areas of fine-textured soils will usually reflect the darkest tones. Areas in which considerable frost activity occurs, as suggested by the presence of many frost boils, may reflect a very light tone if photos are taken after the surface materials have dried.

Cultural features on the airphotos are the reflections of man's activities. Their analysis leads to determination of the value man places on the terrain upon which he is operating. The activity may reflect some degree of adjustment to the terrain thereby becoming, in part, an indicator of materials and conditions. Alterations of arctic - subarctic terrain features, in permafrost areas which are predominantly silty, are often severe. Once the vegetation is destroyed and the thermal regime has been upset, a severe thaw is induced. This condition is easily detected on the photos.

Special features, as a pattern element, are dependent on some inherent quality or characteristic of the material or deposit. They are usually unique and may not be found in association with an unlike material. In arctic - subarctic - polar regions there are many features related solely to the permafrost regime and/or very severe frost activity, and if recognized they become an indicator characteristic.

DIRECT PERMAFROST AND FROST ACTIVITY INDICATORS

Recognition of many permafrost and severe frost activity features is often relatively simple provided that the photo scale and photo quality are such that the minute details are not lost.

In many permafrost areas polygons are among the more outstanding markings easily recognizable on photos of suitable quality and scale, with the exception of areas mantled with a dense forest cover (Figs. 24, 26, and 27). Polygons are direct indicators of the presence of permafrost. Polygons, in general, are confined to unconsolidated materials which have been rendered solid by freezing. In most instances the polygon indicates an undesirable soil situation—one in which there is a high percentage of ice in the soil mass. However, the photo pattern containing polygons must be evaluated in light of topographic position, soil parent material, and drainage since exceptions to the ice-soil relationship associated with the majority of polygons do occur. When they are found in association with silt soils it usually indicates ice either in wedge form or in massive layers depending on the type of polygon found. Polygons with raised centers and a continuous channel as an outlining perimeter contain ice in wedge form beneath the outlining channel. Polygons containing a depressed center and a perimeter consisting of a dike usually occur in association with fine-textured soils in depressed topographic situations. The center portion is usually underlayed by a layer of ice. It is the latter type of polygon which is perhaps the most critical when encountered in engineering construction.

FIG. 25.—POLYGONS ARE AMONG THE POSITIVE INDICATORS OF PERMANENTLY FROZEN SOILS.

FIG. 26.—DEPRESSED CENTER POLYGONS ARE ASSOCIATED WITH ONE OF THE MOST UNDESIRABLE SOIL-PERMAFROST RELATIONSHIPS.

Frost mounds create another of the more outstanding features of the permafrost pattern. The largest ones, often called pingos, occur only in areas of coastal plain soils (Figs. 28 and 29). They are easily recognized by their dome-like or conical appearance and by their topographic situation, which is usually in the central portion of a basin or old lake bed. It is believed that they are the result of upheaval of the surface from subsurface ice pressure, which may reflect sources of water under great pressure below the permanently frozen materials. Many of them contain large cracks in the upper surface. It is not presently known if these reflect an artesian ground-water potential.

The various forms of soil movement related to frost activity and gravity on sloping ground create striking airphoto patterns. In some instances entire hillsides appear to be sliding down the slopes (Figs. 30 and 31). Surface markings include irregularly shaped flows, vegetation stripes extending down the slope, stone stripes, and great lobate or scalloped flows. In the arctic and sub-arctic the processes of solifluction are active agents of erosion and land destruction. The earth runs or flows consist of non-sorted material which is

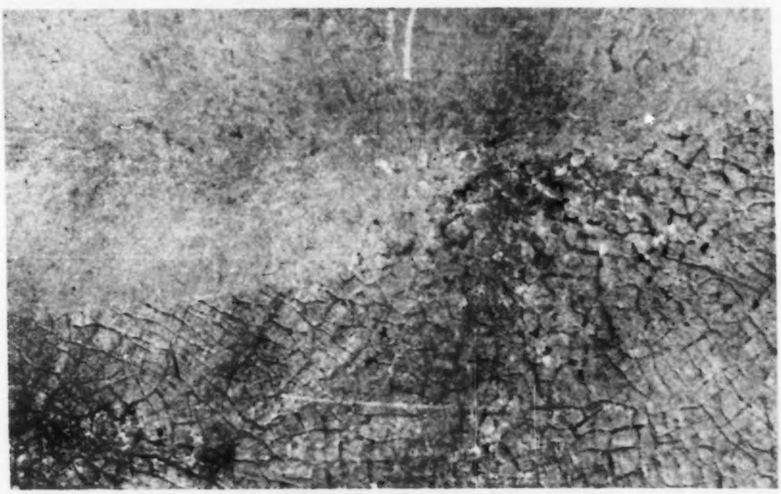

FIG. 27.—VERTICAL AIRPHOTO OF AN AREA CONTAINING RAISED CENTER TYPE POLYGONS.

associated with frost activity and gravity. When a thick mantle of vegetation is present it tends to resist the downward movement until the load of frost-gravity moved rubble becomes too great and a rupture occurs. In areas where vegetation is sparse the soil flows usually extend unbroken for considerable distance down the slope. The channels between the flows appear darker because of higher moisture content and a vegetative cover. The very slight difference in elevation between the flow ridges and the intervening swales affords protection to the plants. Rocky slopes are marked with long streamers of rock slabs and fragments. The foregoing features are easily detected on good quality photos. These markings indicate a very undesirable frost condition—one which may result in severe heave during cold periods, exceptional instability during warm periods, and icing conditions during the winter months. Any use made

FIG. 28.—OBLIQUE PHOTO OF A PINGO IN THE ARCTIC COASTAL PLAIN.

FIG. 29.—GROUND VIEW OF THE PINGO IN FIG. 28.

FIG. 30.—OBLIQUE PHOTO OF A HILLSIDE MARKED WITH SEVERE SOLIFLUCTION LOBES

FIG. 31.—OBLIQUE PHOTO OF AN AREA IN WHICH DOWN HILL MOVEMENT OF SOILS (SOIL STRIPES) IS BURYING A POLYGON AREA.

of hillsides marked with such features must take into consideration the severity of the problem.

Many arctic - subarctic polar areas are marked with minute mounds which are related to frost activity in the mantle or in the active layers (Figs. 32 and 33). In flat areas, the mounds are circular but tend to elongate with changes in slope. Mounds vary considerably in their size, shape, arrangement, and manner or origin. Some are believed to reflect the action of silt soil being forced upward through cracks or holes in the active layer and spreading out over the tundra vegetation. Many bouldery areas are marked with numerous small circular areas of fine-grained soil which appear to have worked up through the mass of boulders. In glacial till areas which are predominantly silty, the entire active zone appears to be constantly undergoing frost action turn over. Boulders appear to be working out of the ground in flat areas. In sloping areas, they are in process of burial from flow from above (Fig. 34). In areas swept by high winds the silty soils are carried away as soon as they reach the surface and dry out, thus, leaving a surface which resembles a "desert pavement." Such situations are misleading as the impression of great granularity is created (Fig. 35).

TYPICAL ANALYSIS

One area in Alaska has been selected for purposes of illustrating the airphoto method of obtaining information about earth surface features. It is illustrated in Fig. 36.

Area I.—Aside from location, the regional environmental factors were obtained from study of a large number of photos assembled into mosaic form. The study area is situated in Northern Alaska, north of the Brooks Range and just west of the big bend of the Colville River. The study area is crossed by the Colville River. From the physiographic standpoint the entire study area lies in an upland plateau which is situated between the mountains to the south and a vast coastal plain to the north. The area is often referred to as "Arctic Piedmont" or "Arctic Plateau." The upland is characterized by gently rolling topography which is everywhere rounded except along the Colville River. A decided east-west trend of major forms is in evidence. The Colville, as it crosses the area, has cut a rather deep valley in the upland. In places, steep-sided bluffs rise several hundred feet from the valley floor. The valley is quite broad in places and contains several terraces situated between the flood plain and the upland proper. The terraces are low topographically and exhibit very low relief.

From a geologic standpoint the area appears to be composed of gently dipping strata of interbedded hard and soft rock which is overlain by an unrelated mantle of varying thickness. It appears that the softer materials occur in the thicker beds. An occasional prominent ridge provides a significant break to the generally rolling landscape. The structure generally strikes east-west. The presence of the unrelated mantle is suggested by the over all softening of upland slopes and by drastic changes in slopes associated with intense erosion.

From the climatic standpoint, the area exists under generally low rainfall conditions in an environment of cold winters, and short, but relatively warm, summers. There are ample indicators which suggest the presence of permafrost which establishes the cold climate. Yet, the area is warm enough in the summer to support some variety of vegetation.

FIG. 32.—OBLIQUE PHOTO OF A HILLSIDE MARKED WITH FROST BOILS AND SOIL STRIPES.

FIG. 33.—DETAIL OF SMALL FROST BOILS IN A SILTY SOIL MANTLE.

FIG. 34.—LARGE BOULDER BURRIED BY SILTY SOIL WHICH HAS MOVED FROM ABOVE THROUGH FROST ACTION-GRAVITY PROCESSES.

FIG. 35.—REMOVAL OF FINE-TEXTURED SOILS BY HIGH WINDS CREATES A "GRAVEL PAVEMENT" SURFACE.

Detailed study of the individual stereoscopic photos provides information about the type of materials in the area, the presence of permafrost, and some concept about ice-soil relationships. Fig. 36 is a copy of one of the contact prints which has been marked to show significant features. First, the area of the single photo has been divided into three major landforms—upland, terrace and flood plain. The upland consists of a portion of a dissected highland (plateau) having, generally, softly rounded slopes. The slopes are broken in places by prominent rock outcrops (R). This combination of features suggests thick layers of soft material and thin layers of a more resistant material. Since there is a repetitive lineal pattern it is probable that the rocks are slightly dipping. The terrace area appears to be composed of two terrace surfaces, one of which is quite broad and the other occurring as scattered terrace remnants (TR). The

FIG. 36.—AIRPHOTO OF STUDY AREA.

upper surfaces are not at the same elevation. Boundary conditions of the terrace have been difficult to locate because of the rather subtle topographic break between terrace and flood plain in places. Separation has been based on other pattern features. The flood plain contains the stream, islands, bars, and other low-lying areas.

The drainage pattern of the area, on a regional basis, is trellis in plan suggesting that the area is composed of layered hard and soft tilted rocks. The second and third order streams, and those which have not cut deeply into the upland, do not appear to be controlled by the rock type and structure. A more random arrangement exists. This suggests the presence of an unrelated mantle

in the rocks. As the upland streams cut through the mantle and reach the rock below, a change in plan and stream profile occurs.

The erosional features reflect the permanently frozen condition of the materials. Upland gullies include placer-type gullies lined with intense polygon nets (GPo), parallel gully systems (GPa), and button type gullies (GB). The parallel gullies discharge on to the terrace and do not continue across the terrace. The button gullies also discharge on to the terrace but extend for a considerable distance outward. The circular pools are larger on the terrace than in the upland position of the same gully system. There is no well-developed surface drainage system on either the flood plain or the terrace.

Tundra-type vegetation prevails throughout. There are no indications that barren conditions prevail. It is probable that the terrace and upland supports grasses, berries, mosses, and dwarf varieties of willow and birch. Larger plants occur along the water courses. Willow, perhaps 4 ft to 5 ft high (Vw), occurs on the better drained granular soils of the islands and bars. Alder thickets (V_A) occur on cold wet soils in the flood plain.

The photo tones of the entire area are, generally, light. Open-water areas are black. Some of the, generally, wet areas are dark grey. Tones, in this instance, are related more to the predominance of light green of the vegetation than to the actual color of the mineral soils.

There are many special or unique features which are in evidence and which lend strong support to permafrost and ice-soil relationship. Permafrost features in the upland include button-type gullies (GB), polygon lined placer gullies (GPo), "geometric" (those whose thread follows a series of polygon channels) gullies (GG), large thermokarst lakes (LT), areas of depressed-center polygons (PD), area of raised-center polygons (PR), and thawing polygon channels (PT). Permafrost features on the terrace include extensive areas of depressed-center polygons (PD), extensive areas of raised-center polygons (PR), button-type gullies on the terrace remnants (GB), large rectangular polygons on higher and more granular parts of the terrace (PR_e), and numerous abandoned meander lakes which are being obliterated by vegetation invasion. Thus, the upland mantle can be expected to be frozen within perhaps 12 in. to 18 in. of the surface. In areas protected by thick moss the active zone may be within 6 in. to 12 in. On the terrace, the light-toned terrace remnants can be expected to be unfrozen for 4 ft to 5 ft during the latter part of the thaw season. One type of adverse soil permafrost conditions are associated with the depressed-center polygons (PD) as the major portion of the upper few feet will contain much ice. Another is that related to channel type polygons (PR) in which ice will occur in wedge form beneath the channels.

From an engineering standpoint the upland soils consist of a semi-granular to non-granular mantle of varying thickness on sandstone and shale. The mantle is shallow in areas having prominent topographic breaks. Local valleys are filled with colluvial soils which are, generally, fine-grained. The most serious permafrost conditions occur on sloping ground, in the upland depressed situations, and on polygon ground. Shallow cuts are to be avoided as they will not intersect rock and severe thaw will occur. Deep cuts which extend to the sandstone and utilize the sandstone for support will be permissible but will be difficult to do because of the disturbance of the surroundings. Runway orientation which is coincident with the long sandstone ridges will offer one feasible solution. All valley-fill type gullies, depressions, and side-hill situations will require fill and should not be disturbed prior to use.

The higher portions of the terrace (TR) consist of frozen sand and gravel with scattered areas of fine-grained soils on the surface. Limited amounts of granular materials are available for borrow when the seasonal thaw has reached a maximum. The remainder of the terrace is low and marshy and contains considerable subsurface ice. The soils are predominantly fine-grained with considerable peat in the upper part. The terrace remnants will offer good sites for buildings. In this location, they are too small for use in runway location. The remainder of the terrace area should not be disturbed. If it is necessary to utilize any portion then end-dump-fill procedures should be followed. The flood-plain soils are granular and contain materials available for construction use.

SUMMARY AND CONCLUSIONS

In concluding this discussion on the use of aerial photographs for obtaining information about earth-surface materials and their condition in arctic - subarctic regions, it is well to point out a few of the more important results and conclusions concerning the present stage of development.

1. Accurate determinations of soils and permafrost conditions employing photo-interpretation techniques is contingent on a background in some of the earth sciences as well as some phases of civil engineering, experience in the arctic and subarctic, cognizance of the relationship between the areal soil patterns and engineering problems, a keen imagination, and the ability to apply processes of logic and deductive reason to the analysis of physical features as reflected in the airphotos.

2. Proficiency in using airphoto analysis techniques makes it possible to distinguish good materials and good site areas from detrimentally frozen materials and unsatisfactory site areas.

3. The presence, degree, and type of permafrost, as well as areas of severe frost susceptibility, can be predicted in most situations in the arctic and subarctic.

4. The general physical characteristics of soil-parent materials as well as materials for engineering construction can be identified and delineated with east in most regions throughout the arctic and subarctic.

5. It is possible to make general determinations concerning whether a material is chiefly sand, gravel, silt, or clay and, in some instances, it is possible to predict combinations of the foregoing. Detailed information about such physical properties as grain size, structure, engineering limits, profile development, and natural densities cannot be obtained by direct photo analysis methods.

6. It is not possible to determine the suitability of granular material for aggregate for Portland cement concrete or bituminous concrete use.

7. In many instances it is possible to predict the presence and type of overburden on granular materials which may be considered for construction use.

8. In areas where rock is being considered for use as a source of engineering materials the following information can often be obtained from airphotos: lithological and structural characteristics; degree and type of weathering and presence of an overburden; landslide potential; trafficability condition of both the site and access routes to the site; and presence of talus materials.

9. When talus material is being considered, proper use of the method will often make it possible to distinguish between those materials which are dry and unfrozen and can be excavated, and those materials which contain considerable fines in their mass and which are frozen and cannot be excavated by normal procedures.

10. It is not possible to predict the amount of material, either rock or granular, expected to be obtained from a quarry or pit.

11. From a trafficability standpoint, the trafficable and non-trafficable areas can be identified and delineated for heavy construction equipment use.

12. In undeveloped and unmapped regions airphotos can be used to great advantage in all activities pertaining to airfields, highways, and other engineering works by making general engineering soils and drainage maps which show good and bad soils based on anticipated performance.

13. A materials survey effected through the use of airphotos can be done more rapidly, more efficiently, more accurately, and at a greater savings in cost than by use of any other method.

14. The principles as developed in the temperate climates have found considerable application in the arctic and subarctic and it is believed that they can be applied to any other region, regardless of climate, where engineering studies involving the natural and physical features are concerned.

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EPOXY ASPHALT CONCRETE FOR AIRFIELD PAVEMENTS[a]

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SYNOPSIS

Studies of airfield-pavement problems which arise from the combined effects of heavy load, fuel spillage, and jet blast have led to the development of a new type of paving material called epoxy asphalt concrete (EAC). Conventional hot-mix asphalt plants and paving equipment are used in its production.

This material has Marshall stability values of 15,000 lb to 20,000 lb and is capable of withstanding tire pressures in excess of 1,000 psi. In repeated gyratory loading simulating B-52 traffic, EAC resists densification and retains its load-carrying ability. Good retention of strength and stability under severe conditions of solvent and fuel spillage has been established by experience in aircraft maintenance areas. In jet-blast tests with military planes, EAC performs well under normal pretakeoff conditions and with prolonged afterburner operation producing pavement temperatures of 800°F.

Investigation of its mechanical properties shows that EAC has a flexural strength or modulus of rupture exceeding that of portland cement concrete by a factor of 3 to 6. However, EAC has flexibility which allows it to be bent to the same extent as asphaltic concrete before fracture. The new paving material thus combines the strength of portland cement concrete with the flexibility of asphaltic concrete.

Note.—Discussion open until October 1, 1960. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Air Transport Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. AT 1, May, 1960.

[a] Presented at the October 1959 ASCE Convention in Washington, D.C.

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[4] Shell Development Co., Emeryville, Calif.

INTRODUCTION

The rapid growth of both military and commercial aviation has been accompanied by ever-increasing aircraft weight, higher wheel loads, and higher tire pressures which have posed many structural problems in the design and construction of adequate pavements. To these structural problems have also been added some special requirements which have accompanied the development of jet aircraft: these are good resistance to the high temperatures and erosive effects of jet blast, and the ability to tolerate fuel spillage. The nature of these problems has been investigated by a number of military and commercial organizations and has been well presented at the Jet Age Airport Conference held in New York, N. Y.[5] Thorough discussion of these problems has also taken place at three United States Air Force Pavement Conferences[6] which have brought together representatives of military and civilian aviation groups, military personnel responsible for the design and construction of airfields, and representatives of various paving industry groups. From a study of the papers presented at these conferences it is clear that neither rigid nor flexible pavements of the conventional type offer a complete solution to the combined problems of very heavy load, high temperature, high velocity blast, and fuel spillage. Any new paving material which can provide performance under these conditions which is superior to that obtainable with conventional pavements should find a place in airfield pavements of the future.

A NEW PAVING MATERIAL

From research on a large number of new paving compositions, epoxy asphalt concrete (EAC) has emerged as a material well suited to solve simultaneously the problems of heavy loads, high temperature and velocity blast, and fuel spillage. It is a combination of a graded mineral aggregate and EPON* asphalt binder. The properties of the pavement are determined by the properties of the binder which contains epoxy resin well-known for its outstanding performance as an adhesive and for its remarkable physical properties. Initially the binder is a black, viscous liquid hardly distinguishable from asphalt. During mixing with hot aggregate, subsequent hauling, laying, compaction, and field curing, the binder is converted by a chemical reaction from a viscous liquid into a non-melting elastic polymer with interesting mechanical properties and solvent resistance. The rate of the polymerization reaction taking place is readily controlled so that the aggregate-binder system can be handled in normal hot-mix asphalt plants and can be spread and compacted by conventional paving machines and rollers.

The finished pavement has the appearance of normal asphalt concrete but the properties are very different in many respects. Initial applications of epoxy asphalt concrete have been in the form of overlay pavements $\frac{1}{2}$ in. to 1 in. thick which have been used as a protective layer for existing flexible and rigid pavements. Studies are under way which should lead to a design for the use of epoxy asphalt concrete as the major pavement element in new construction.

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[5] Journal of the Air Transport Division, Proceedings, ASCE, December, 1957.

[6] Roads and Streets, September, 1959, p. 111.

* Trademark, Shell Oil Co.

EPOXY ASPHALT BINDER

The binder is composed of a liquid epoxy resin and a paving grade asphalt containing an additive which acts as a flexibilizing coreactant. When these ingredients are mixed in the proper proportions, a chemical reaction begins which causes the viscosity to rise steadily until the binder is converted into a thermoset plastic. The rate of the reaction and the corresponding rise in viscosity are determined by the temperature of the system. The reaction may be caused to proceed at a rapid rate by choosing a high temperature or at a slow rate by choosing a low temperature. The operating temperature range for practical handling of the binder in paving applications is about 200°F to 300°F and includes the preferred operating range of hot mix paving plants.

When the binder is cured in the form of sheets it may be tested in tension according to ASTM method D412-51T. Dumbbell-shaped specimens stamped from the sheet and tested at a rate of 20 in. per min at room temperature show a tensile strength of about 1,000 psi and an elongation at break of 200% to 300%. Correcting the tensile-test value for the change in cross-sectional area during the test gives a true tensile stress at fracture of about 3,000 psi. The binder is elastic and will return to its original shape after deformation by twisting or stretching.

The binder is particularly well suited to high-temperature use as shown by the fact that it does not melt or lose its shape when heated to 800°F on a hot plate. Heating to a high temperature does not destroy the good low temperature flexibility of the binder as indicated by the results of the Fraass test shown in Table 1. In this test (Institute of Petroleum method 80/53) a 0.5 mm thick layer of asphalt or other product is placed on a spring-steel strip and subjected to periodic bending which produces a maximum tensile strain of 0.03 in. per in. over a period of 11 sec. The temperature is gradually lowered until

TABLE 1.—LOW TEMPERATURE FLEXIBILITY OF EPOXY ASPHALT AS SHOWN BY THE FRAASS TEST

	Initial	After 5 Min at 760°F
60 Pen Asphalt	+16°F	+25°F
60 Pen Epoxy Asphalt	-31°F	-31°F

the specimen fractures under this strain. Table 1 shows that a 60 penetration paving asphalt fractured at +16°F in this test while epoxy asphalt binder failed to fracture at -31°F. After exposure to a temperature of 760°F for 5 min the Fraass breaking point of the asphalt was raised to +25°F while the epoxy asphalt still failed to fracture at -31°F.

PREPARATION AND PLACEMENT OF MIXES

Epoxy asphalt concrete is prepared in conventional hot-mix asphalt plants, laid with self-propelled paving machines or by hand spreading, and compacted with conventional steel and rubber-tired rollers. Construction specifications

for successful application of this new product have been established in a series of field installations conducted during the past 3 yr using commercial hot-mix plants and paving equipment.

Mix Plant Operation.—Minor additions to the mix plant are required to provide for pumping of the epoxy resin and additive to the weigh platform where blending of the binder components is accomplished in the weigh bucket with brief stirring. Alternatively, the additive is preblended with the asphalt and only the epoxy resin is added in the weigh bucket. Laboratory and field studies of mix design have shown 8% to 10% weight of binder basis aggregate to be the preferred range for preparation of dense, flexible, and impermeable epoxy asphalt concrete. Asphaltic concrete mixes containing 8% to 10% binder would have very poor stability at the high pavement temperatures encountered in warm climates and under jet blast because the asphalt is essentially a viscous liquid at high temperatures. Since epoxy asphalt is a non-melting plastic which retains its shape at high temperatures it is possible to obtain flexible mixes with high binder contents without the serious loss of stability usually experienced with asphalt.

Aggregate complying with conventional quality and gradation specifications is used, and for overlay pavements $\frac{3}{4}$ in. to 1 in. in thickness, the maximum aggregate size is set at $\frac{1}{4}$ in. Dense graded aggregates are preferred where dense, impermeable pavements are desired to act as a protective overlay for asphaltic concrete against attack by various fluids. Aggregate temperature is an important item in successful handling of epoxy asphalt concrete since the rate of the chemical reaction taking place in the binder, and hence the binder viscosity, is determined by the temperature of the mix.

Timing the Paving Operations.—The mix temperature has an important influence on the time available for subsequent operations involved in laying the pavement. At high temperatures the polymerization reaction proceeds more rapidly, and the viscosity of the binder increases more rapidly than at low temperatures. The mixing of binder and aggregate presents no problem because the viscosity of the binder remains for about 30 min within the viscosity range in which penetration grade asphalts are normally mixed with aggregate. Normal mixing time in the pugmill is required, usually 1 min or less. Laboratory and field experience have shown that the mixing, hauling, and spreading of the mix should be accomplished by the time the binder viscosity has reached 60 poises in order to be able to obtain the desired degree of compaction with conventional rollers. The aggregate temperature to be used in any given construction job will be determined ordinarily by the hauling time involved, since the mixing time in the plant and the time required for spreading are essentially constants. Where the hauling time is short, relatively high aggregate temperatures can be used, but for long hauls lower temperatures are required.

Surface Preparation.—When epoxy asphalt concrete is used as an overlay on existing pavements, the surface should be swept free of dust and dirt and a suitable tack coat should be applied. Tack coats of emulsified asphalt, cutback asphalt, and penetration grade asphalt have been used successfully in many installations. Where the pavement is subjected to normal pretakeoff conditions by military or commercial jet aircraft, an asphalt tack coat appears adequate. In some cases where the overlay is placed on portland cement concrete and is subjected to prolonged high-temperature blast, such as is maintenance and engine overhaul areas, a tack coat of epoxy asphalt binder has been successfully used.

Compacting the Mix.—Laboratory studies and examination of field installations have shown that low void content of the compacted mix is very beneficial in obtaining the impermeability and high solvent resistance desired in epoxy asphalt concrete overlay pavements. The retention of strength after long exposure to jet fuel as measured by a punching shear test is shown in Fig. 1. Here it may be seen that a void content less than 4% based on saturated surface dry specific gravity of the aggregate is required for high strength retention.

Rolling studies at field installations, using steel rollers weighing from 3.5 tons to 12 tons, have shown that the desired density and low void content can be obtained by making the break-down roll with 10 ton or 12 ton steel rollers followed by surface finishing with a rubber-tired roller with a weight of 2,000 lb per wheel and 90 psi tire pressure.

PROPERTIES AND PERFORMANCE OF EPOXY ASPHALT CONCRETE

To determine the potentialities of this new paving material, a program has been undertaken in which mechanical properties have been investigated in the laboratory, and performance under conditions of actual use has been determined in the field. The field installations, consisting of $\frac{1}{2}$ in. to 1 in. thick overlay pavements, have been placed and studies have been made with mixes using dense-graded aggregates with a maximum particle size of $\frac{1}{4}$ in.

Stability and Bearing Capacity.—As mentioned earlier, a chemical reaction takes place in the binder which converts it from a viscous liquid to a nonmelting plastic. This reaction is still in progress when the hot mix is spread and compacted, and continues for some time in the pavement. A convenient method of following the progress of the reaction in the pavement is to determine the bearing capacity of the pavement periodically. The 90° cone penetrometer test commonly used in soil test work has been shown[7] by L. W. Nijboer to give bearing capacity results in good agreement with the theory of L. Prandtl[8] when used on sand sheet mixes of the type involved in epoxy asphalt overlays. The bearing capacity measured is equivalent to the tire pressure which can be tolerated. Fig. 2 shows data taken on an EAC overlay placed at an aircraft maintenance base in the San Francisco, Calif., area. This overlay was able to support a truck with a 17,700 lb rear axle load without shoving within 15 min after compaction with steel rollers. From Fig. 2 it can be seen that 1 day after placement, the pavement could withstand a tire pressure of 200 psi and this increased to 740 psi within 1 week. A bearing capacity of 1,000 psi was reached within 10 days and values in the range 2,000 to 3,000 were attained in 30 days. Air temperatures during this period were in the range 55°F to 75°F and pavement temperatures did not exceed 105°F. At higher ambient temperatures a more rapid rise in bearing capacity is obtained.

By means of the Marshall test the stability of epoxy asphalt concrete has been determined at several levels of binder content. For comparison, asphaltic concrete specimens were prepared from the same crushed aggregate with 85/100 penetration grade asphalt as the binder. The specimens were prepared

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7 "Plasticity as a Factor in the Design of Dense Bituminous Pavements," by L.W. Nijboer, Elsevier Publishing Co., Inc., New York, p. 109.

8 "Concerning the Hardness of Plastic Bodies," by L. Prandtl, Nachrichten von der Königlichen Gesellschaft der Wissenschaften zu Göttingen, Math. Phys., 1920, p. 74.

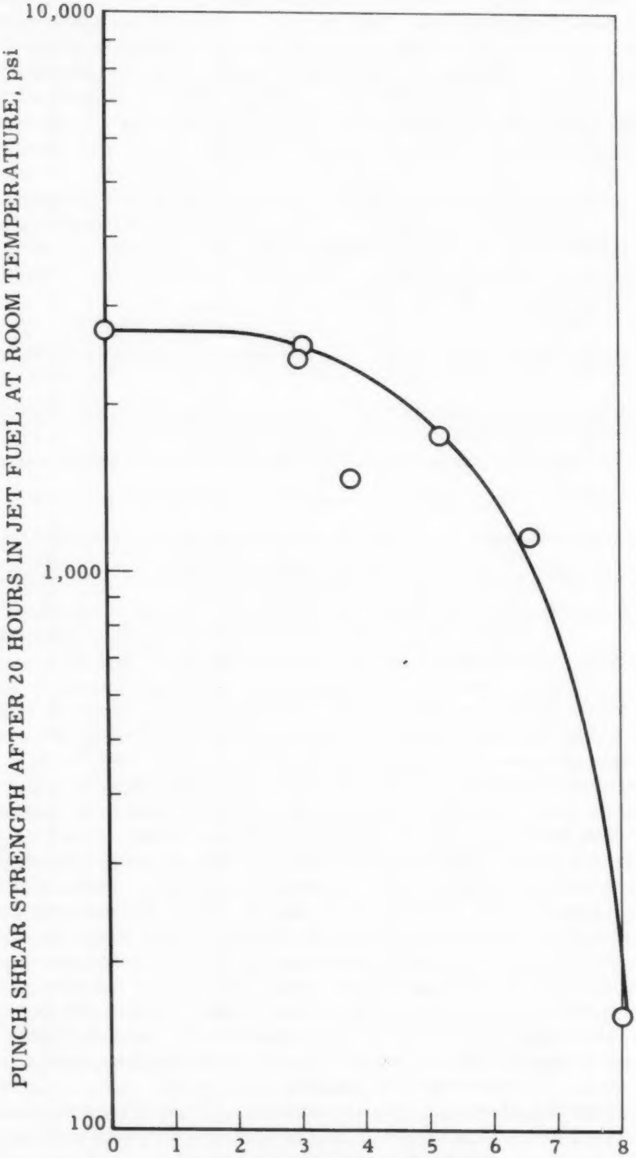

FIG. 1.—INFLUENCE OF AIR VOID CONTENT ON PUNCH SHEAR STRENGTH AFTER 20 HR OF IMMERSION IN JET FUEL AT ROOM TEMPERATURE

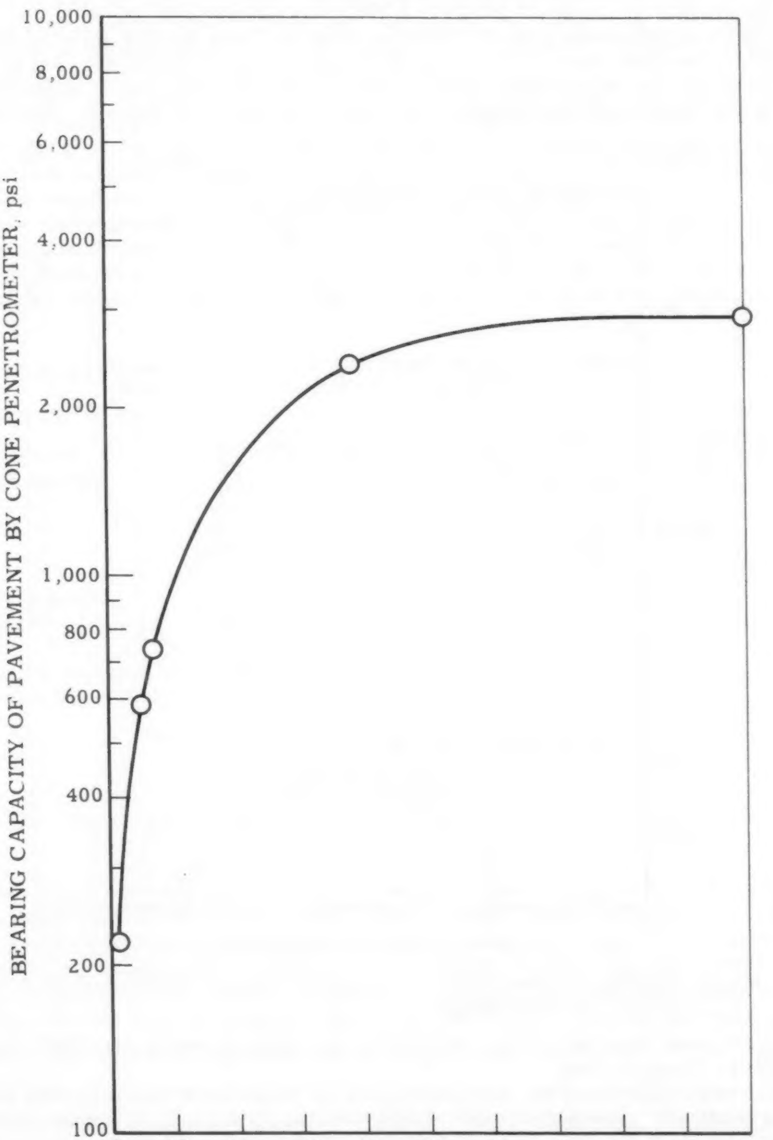

FIG. 2.—INCREASE OF BEARING CAPACITY WITH TIME

with 75 blows on each face and were cured 4 hr at 250°F. The results given in Fig. 3 show the stability of EAC to be in the range of 15,000 lb to 20,000 lb as compared with a range of 800 lb to 3,500 lb for conventional asphaltic concrete made from this dense-graded crushed aggregate. There is little apparent damage to the EAC specimens during Marshall testing, and when the load is removed the specimen rebounds, about 60% to 70% of the "flow value" actually being an elastic and recoverable deformation as shown in Table 2. Asphaltic

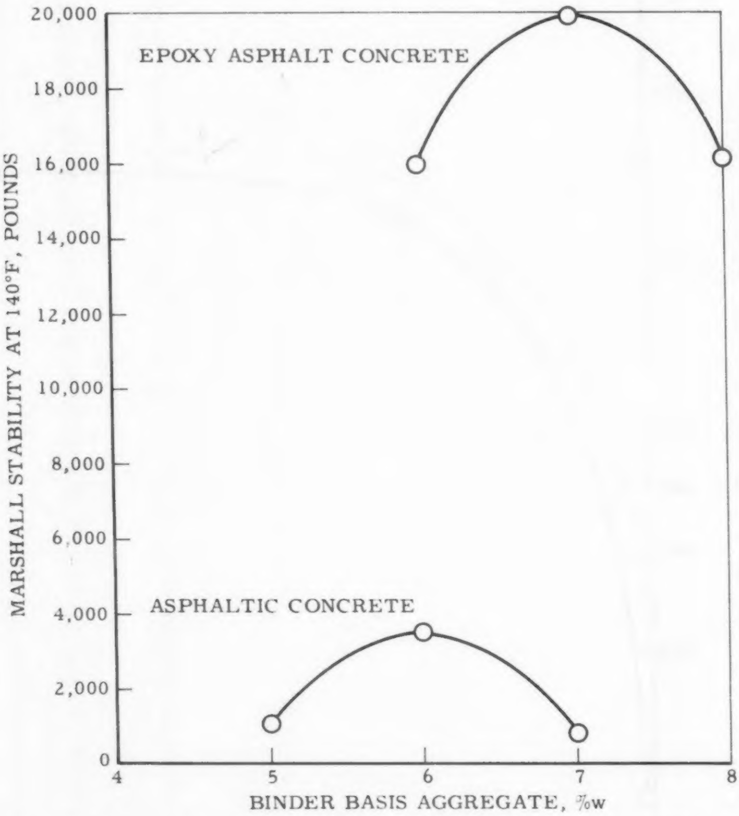

FIG. 3.—MARSHALL STABILITY OF EPOXY ASPHALT CONCRETE AND ASPHALTIC CONCRETE

concrete does not show this type of recovery but remains permanently deformed after the Marshall test.

A recent analysis of the Marshall test by C. T. Metcalf[9] has led to the development of a procedure for calculating the bearing capacity in terms of tire pressure which the pavement can tolerate at 140°F without the production of a compressive strain exceeding 1%. The values of bearing capacity calculated by this new procedure correlated well with field experience in 84 out of 91

[9] "Use of the Marshall Stability Test in Asphalt Paving Mix Design," by C.T. Metcalf, Highway Research Bd. Bulletin, No. 234.

pavements studied by Metcalf. In Table 2 the bearing capacities of epoxy asphalt comcrete are shown to be in the range of 700 psi to 1,100 psi as compared with 250 psi for the asphaltic concrete made with the same aggregate. This represents an improvement which is of particular significance in view of the high tire pressures and wheel loadings which are encountered in modern military aircraft.

The ability of pavements to retain high load-carrying capacity after many repeated loadings is particularly important in taxiways and runways where aircraft traffic is channelized. Repeated loading of epoxy asphalt concrete simulating thousands of coverages by traffic has shown that epoxy asphalt concrete retains high stability and low flow values and resists densification.

The bearing capacity values mentioned previously are far beyond the requirements of aircraft now (1960) in use, the highest tire pressures currently encountered being about 500 psi on some aircraft of the United States Navy. A practical example of the need for the high bearing capacity of EAC is found in

TABLE 2.—MARSHALL STABILITY AND FLOW VALUES FOR EPOXY ASPHALT CONCRETE AND ASPHALTIC CONCRETE AT 140°F

% Binder Basis Aggregate	Average Marshall Test Values from Duplicates				Bearing Capacity,[a] in psi
	Stability, in pounds	Flow, in Inches			
		Total	Rebound	Permanent	
6	15,900	0.17	0.12	0.05	973
7	19,900	0.18	0.12	0.06	1135
8	16,100	0.22	0.13	0.09	706
6% Asphalt 85/100	3,560	0.14	0	0.14	275

[a] Tire pressure which can be tolerated at 140°F without producing a compressive strain in excess of 1%:

$$\text{Bearing Capacity} = \frac{\text{Stability}}{5 \times \text{Flow}} (K + 2)\ (1 - 0.055\ K)$$

$$\text{where } K = \frac{1 + \sin\ (59.4 - 0.942\ \text{Flow})}{1 - \sin\ (59.4 - 0.942\ \text{Flow})}$$

areas where vehicles such as fork-lift trucks and steel-wheeled dollies carrying heavy loads on small diameter wheels are used. Such a case was encountered in an aircraft assembly plant where large sections of structural members were being transported on trains of steel-wheeled dollies with wheel loads of 800 lb per in. of wheel width. The rough, cracked surface of the portland cement concrete floor was causing damage to the aircraft sections and this condition was alleviated by placement of a $\frac{1}{2}$ in. EAC overlay to serve as a new smooth-running surface. The overlay was placed during a weekend and had developed sufficient bearing capacity by the following Monday to support the heavy loads without indentation of the pavement.

Fuel and Solvent Resistance.—The resistance of epoxy asphalt concrete to deterioration by jet fuel and various solvents is an important property in many applications where the overlays are used to protect underlying pavements. To

prevent fuels and solvents from seeping through the overlay it should be impermeable. This condition is achieved by the use of a dense-graded aggregate and relatively high binder content in the mix, and by the use of heavy rollers in the compaction operation. The solvent resistance of the overlay itself is also improved by high densities and low air void content as shown in Fig. 1, which shows the results obtained on cores from field installations. The resistance of $\frac{1}{2}$ in. thick specimens to the shearing action of a $\frac{7}{16}$ in. diameter steel rod driven at a rate of 0.2 in. per min by a testing machine is designated as the punch shear strength. The punch shear strength of specimens after 20 hr of immersion in jet fuel at room temperature decreases as the air void content increases. On the basis of data of this type from a number of field installations, an air void content of 4% (based on saturated surface dry specific

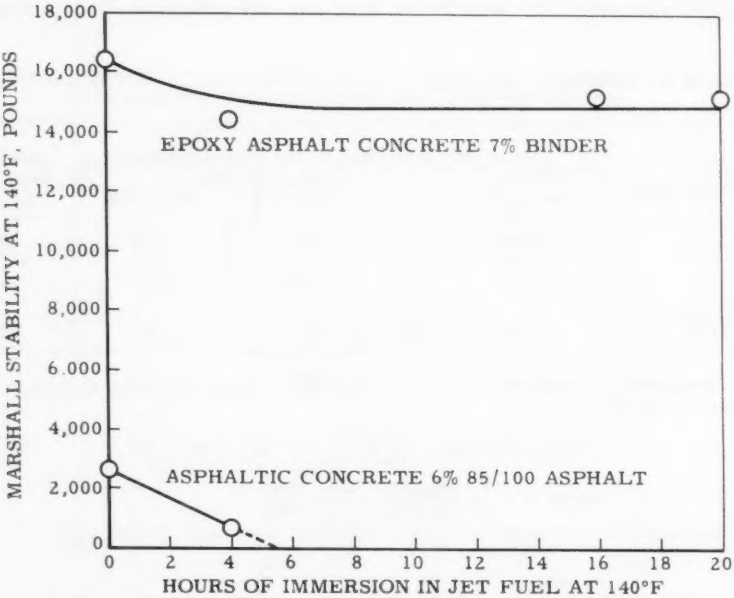

FIG. 4.—SOLVENT RESISTANCE OF EPOXY ASPHALT CONCRETE COMPARED WITH ASPHALTIC CONCRETE

gravity of the aggregate) has been set as a goal in the construction of overlays where solvent resistance is important. The relative solvent resistance of low air void content asphaltic concrete and epoxy asphalt concrete to jet fuel at 140°F is shown in Fig. 4. Asphaltic concrete disintegrates within 6 hr under these severe conditions while EAC retains a Marshall stability of about 15,000 lb.

The solvent resistance of epoxy asphalt concrete pavements has been tested under severe conditions in field installations. One of the early installations was in a bulk depot in which fuels and lubricants are loaded into tank trucks. A small amount of spillage and dripping occurs constantly and frequent replacement of asphaltic concrete was required. A $\frac{1}{2}$ in. overlay of EAC has been

effective in protecting the underlying pavement from softening by fuels and lubricants for several years.

An extremely severe condition of solvent and fuel damage was encountered at the maintenance base of a commercial airline in the San Francisco area. As part of the maintenance and overhaul routine, planes are placed on a designated area where engines soiled by oil leaks are washed down with a petroleum solvent, resembling paint thinner, which falls on the pavement. Oil filter changes are made with some spillage; fuel tank drains at eighteen locations on the aircraft are opened to remove water and some highly aromatic aviation gasoline as well. Hydraulic fluids of both the oil base and alkyl aryl phosphate type (such as Skydrol) frequently accumulate in small amounts on the pavement. As a result of outstanding performance of an EAC overlay in this severe service, the airline elected to protect a 3-acre area of asphaltic pavement at a new jet maintenance base with an epoxy asphalt overlay.

Jet Blast Resistance.—Laboratory tests have been used to obtain an indication of the strength of epoxy asphalt concrete at high temperatures but no means has been found in the laboratory to simulate the combined high temperature and high-velocity blast of jet engines. Actual field performance under blast from jet aircraft has therefore been used to evaluate epoxy asphalt concrete. Epoxy asphalt overlays 1-in thick have been placed on spalled portland cement concrete slabs in overhaul areas used for jet planes at two military air bases in California. Thermocouples have been installed in the overlays for temperature measurements during the blast tests. These tests used a cycle which simulated normal pretakeoff operation and consisted of periods of idle power operation and of 100% power or "military" operation. In other tests prolonged use of the afterburner was also included which resulted in surface pavement temperatures as high as 800°F. A survey reported by W. J. Turnbull and C. R. Foster[10] shows that the use of afterburners occurs only on planes already in motion, with the exception of maintenance operations which are carried out in special areas such as the one in which these tests were made.

The overall performance of the overlay in these tests, which involved conditions far exceeding the severity and duration expected in normal use by military aircraft was judged to be excellent. Detailed results of these tests may be released by the military agencies at a later date. It has been pointed out[11] by J. H. Litchfield that the severity of jet blast and temperatures encountered with commercial airline jet planes is considerably less than in the case of military aircraft.

Tensile Properties.—In order to predict the behavior of epoxy asphalt concrete under various conditions of loading at various temperatures, a knowledge of its major mechanical properties is required. The tensile strength has been determined over a temperature range of 32°F to 140°F at loading times from about 1 to 10^6 sec as shown in Fig. 5 for EAC with 8% binder in the mix. The data for loading times up to about 5,000 sec were obtained on a conventional testing machine and those for longer times were produced in constant-stress creep-rupture tests. The tensile strength is rather insensitive to changes in loading time and in this respect resembles portland cement concrete much more than asphaltic concrete. Changes in tensile strength with temperature are also considerably less than those found with asphaltic concrete.

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[10] "Effects of Jet Blast and Fuel Spillage on Bituminous Pavements," by W.J. Turnbull and C.R. Foster, Journal of the Air Transport Division, Proceedings, ASCE, Vol. 85, 1959.

[11] "Effects of Jet Fuel Spillage and Blast on Pavements," by H.J. Litchfield, Journal of the Air Transport Division, Proceedings, ASCE, Vol. 85, 1959.

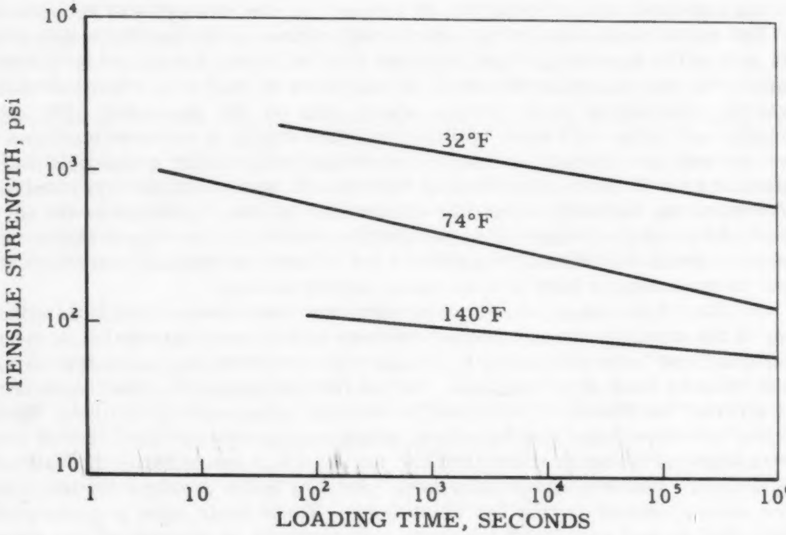

FIG. 5.—TENSILE STRENGTH OF EPOXY ASPHALT CONCRETE AS A FUNCTION OF TEMPERATURE AT LONG LOADING TIMES

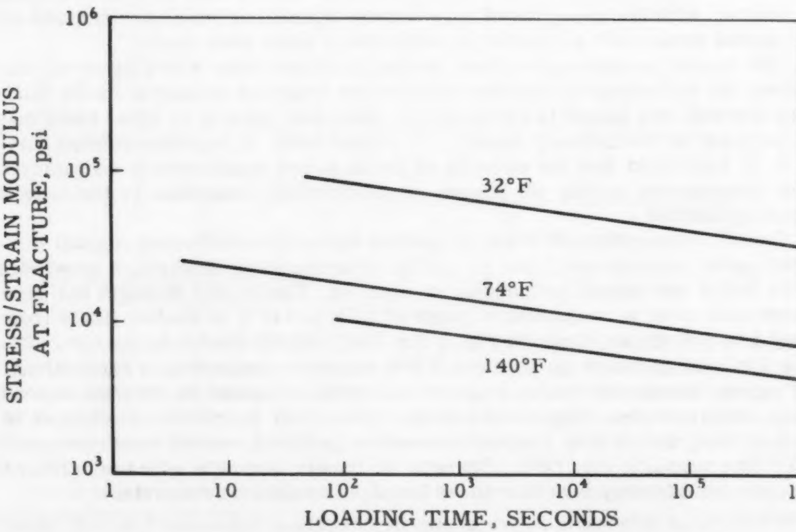

FIG. 6.—STRESS-STRAIN MODULUS OF EPOXY ASPHALT CONCRETE AS A FUNCTION OF TEMPERATURE AT LONG LOADING TIMES

The tensile strain of epoxy asphalt concrete at fracture proved to be almost independent of the loading time and averaged about 3%. This tensile strain is about three times that obtained with asphaltic concrete and about ten times the tensile strain at fracture of portland cement concrete.

From the tensile strength and strain measurements, the stress-strain modulus at fracture has been determined over a wide range of loading time and temperature as shown in Fig. 6 for EAC with 8% binder in the mix. This type of information is useful in determining the response of the material to applied stresses and in calculating the magnitude of stresses which develop in the material as a result of temperature changes. For the latter type of stress calculations the linear thermal coefficient of expansion is also required. This was found to be about 20×10^{-6} in. per in. per °F which is very close to the linear thermal coefficient of expansion of asphaltic concrete.

The response of EAC to dynamic loading of the type associated with moving traffic has been determined by studying the bending of beams of the material at various frequencies corresponding to loading times of 10^{-3} to 1 sec and temperatures from 32°F to 140°F. As is shown in Fig. 7 the stress-strain modulus of the material under these short loading times varies from about 3×10^6 psi to about 10^5 psi in the temperature range 32°F to 140°F. Under these conditions, epoxy asphalt concrete behaves somewhat like portland cement concrete which has a stress-strain modulus of about 3×10^6 psi.

Beams of EAC have been flexed 100×10^6 times without fracture, indicating that the material has good resistance to failure by fatigue.

Flexural Strength.—The flexural strength or modulus of rupture of pavements is an important property which characterizes their behavior under loading. This measurement has been made according to the procedure specified in ASTM methods C293-57T or C78-49 which consists of supporting a beam of standardized dimensions near the ends and producing bending by center loading at a specified rate with a testing machine. From the maximum load and the dimensions of the specimen, the modulus of rupture or flexural strength is calculated. As an addition to the ASTM procedure it was found useful to measure the deflection of the beams at the center during the test by means of a dial gage. This measurement gives a good impression of the relative flexibility of various paving materials.

In Fig. 8 are shown modulus of rupture and beam-deflection values at maximum load for portland cement concrete, asphaltic concrete, and epoxy asphalt concrete. The beams used were 2 in. by 3 in. by 12 in. with a testing span length of 9 in. At the left of the graph the modulus of rupture of portland cement concrete is shown covering a range of about 400 psi to 800 psi. The center deflection of the beams at maximum load averaged 0.01 in. in these tests. Along the bottom of the graph are the data for asphaltic concrete at three levels of binder content with the modulus of rupture in the range 75 psi to 100 psi. The beam deflections are very large because of the flexible nature of asphaltic concrete.

The three center curves in Fig. 8 show the properties of epoxy asphalt concrete at three levels of binder content—7%, 8%, and 12% basis aggregate. The circles, triangles, and squares represent the use of three flexibilizing additives which differ in composition. The modulus of rupture of epoxy asphalt concrete may be varied from about 400 psi, which is about the lower level for portland cement concrete, to about 2,400 psi, which is two to three times that obtained with the best portland cement concrete. At the same time, epoxy asphalt concrete may be flexed to the same extent as asphaltic concrete at maxi-

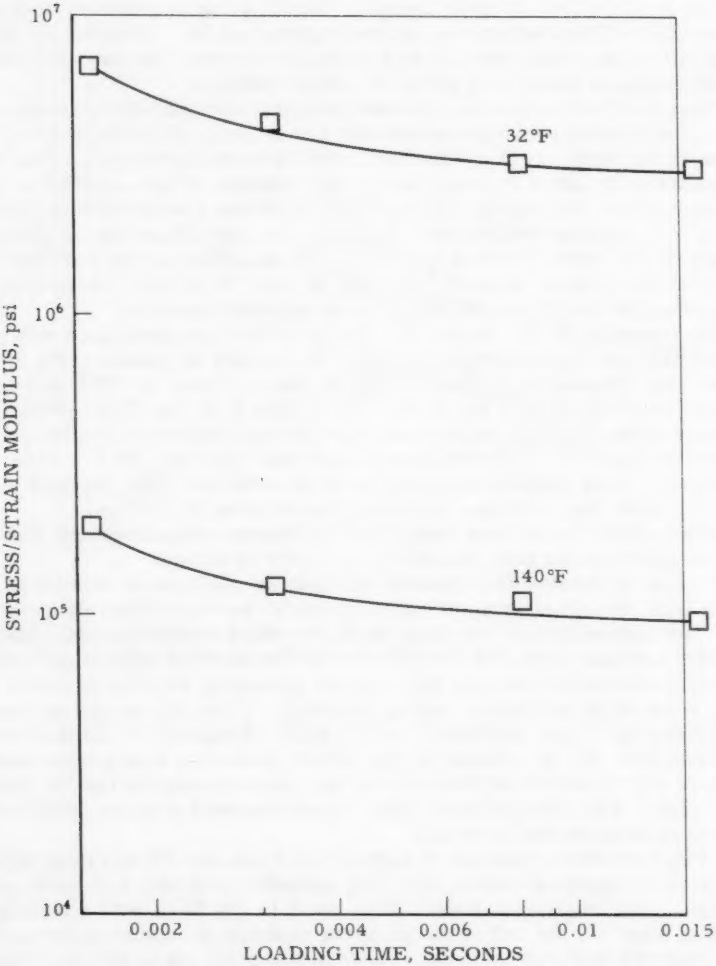

FIG. 7.—STRESS/STRAIN MODULUS OF EPOXY ASPHALT CONCRETE AT SHORT LOADING TIMES

mum load. Thus we have a structural material which combines the desirable properties of both rigid and flexible pavements: a strength equal to or greater than that of portland cement concrete and a tolerance for bending equal to that of asphaltic concrete.

AVAILABILITY OF TECHNICAL ASSISTANCE

The successful use of epoxy asphalt concrete requires proper handling of the material in the mix plant and during the laying operations. The changes from conventional hot-mix handling practices are few but important, and have

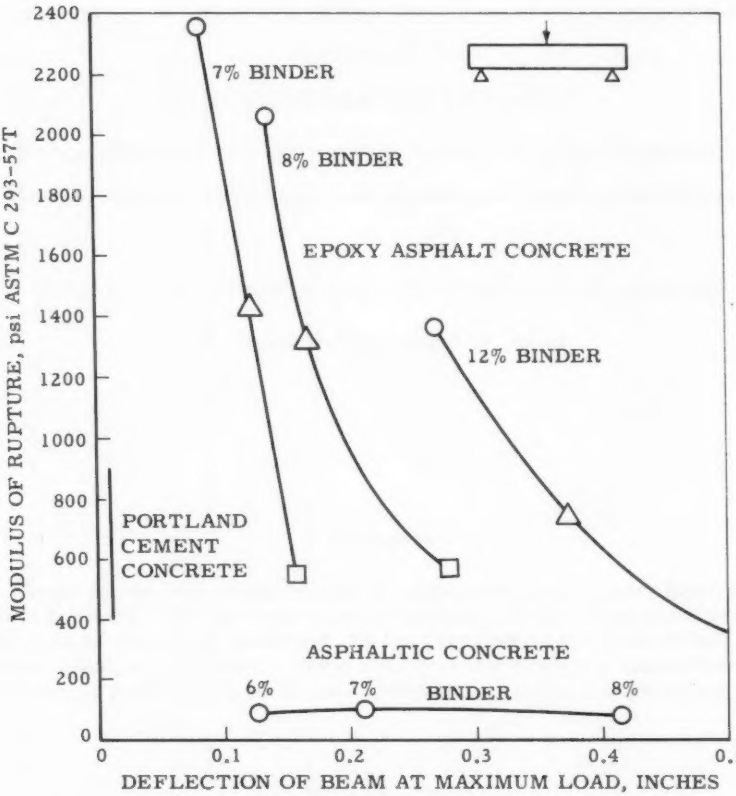

FIG. 8.—MODULUS OF RUPTURE AND FLEXIBILITY OF EPOXY ASPHALT, PORTLAND CEMENT, AND ASPHALTIC CONCRETES

been learned from field experience with commercial mix plants and paving equipment. This experience is available in the form of technical assistance in solving paving problems where the properties of epoxy asphalt concrete indicate its use to be desirable.

ACKNOWLEDGMENTS

The writers gratefully acknowledge the participation of the Products Application and Manufacturing-Research Departments of Shell Oil Company in the field development work. We also wish to acknowledge the assistance of D. C. Whitely, H. Johnson, C. E. Creely, and P. R. Chong in the experimental work.

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Journal of the
AIR TRANSPORT DIVISION
Proceedings of the American Society of Civil Engineers

AIRPHOTO INTERPRETATION FOR AIRFIELD SITE LOCATION

By James H. McLerran,[1] M. ASCE

SYNOPSIS

The principles and procedures of airphoto interpretation are briefly considered. To illustrate the application of airphoto interpretation to airfield site selection, an area (Martinsburg, W. Va.) has been selected and an airfield-site-location analysis prepared. From a study of the airphotos, three sites were selected for preliminary study and a comparative analysis is presented.

INTRODUCTION

The use of aerial photography in engineering planning is not new; many engineers and engineering firms use photogrammetry to obtain the required topographic mapping of a given area at the most economical cost. Information is now fed from the plotter into an electronic computing machine to come up with earthwork quantities. These engineers will be quick to claim that they have adopted modern methods and can do a better job at less cost, but have they made complete utilization of this tool, aerial photography? Many engineers have not.

There are engineers that are becoming aware of another use for aerial photography. This group is aware of the fact that before designing a major engineering project there is need of a well-planned comprehensive study of all factors that will effect the cost of the project. They realize that early in the

Note.—Discussion open unitl October 1, 1960. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Air Transport Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. AT 1, May, 1960.

[1] Asst. Chief, Photographic Interpretation Research Branch, U.S. Army Snow Ice & Permafrost Research Establishment, Corps of Engrs., Wilmette, Ill.

planning phase one must gather regional information and then particular information about all possible sites. Then a comparative study of these possible sites can be made.

Aerial photography is an efficient tool for study of the regional geography and physiography. It is the most efficient way of studying two or three sites in detail for use in preparing a comparative study. That is, it can be used during the planning stage to make an analysis of the soils and other environmental factors that will provide the basis of comparison between alternate sites. Many factors that affect the planning, design, and construction of an engineering project may be determined by a careful study of the airphotos.

The use of airphoto-interpretation techniques to evaluate soil and materials is not new, but has been developed over a period of 15 yr to 20 yr. This has been largely on a research basis, but a few progressive highway organizations and contractors have used such methods for several years with excellent results.

The purpose of this paper is to discuss airphoto interpretation and its application to airfield engineering. First the principles and procedures will be discussed and then an area will be analyzed as an example of the procedure and the information that can be gained.

APPROACH

Principles.—The principle of airphoto interpretation has been explained many times and will be cited only briefly herein. Several references in the bibliography explain the principles and the techniques in detail. The basic principle of the technique is that like materials under similar topographic and climatic environments will exhibit similar airphoto patterns, and unlike materials will present different patterns.

The pattern can be described as the combination of certain recognizable elements as recorded upon the airphoto. These elements that singly or together form a significant pattern are: landform, drainage system, erosion characteristics, photo tones, vegetation, and land use. The basic principle and pattern concept are illustrated in Figs. 1 and 2. Similar materials under similar climatic conditions, but separated by a distance of about 100 miles, are shown in Fig. 1; the two photographs represent patterns of residual soils over limestone (note the similarity in landform, drainage, and land use). Dissimilar materials under similar climatic conditions in southern Indiana provide contrasting airphoto patterns as seen in Fig. 2. Fig. 2(a) illustrates the airphoto pattern (sinkhole pattern) developed in areas where flat-lying limestone and its residual soils are exposed to a semi-humid climate. Here the soils, consisting of plastic clays, will be found to be 6 ft to 10 ft deep and only infrequent, shallow rock excavation will be necessary for airport construction. Fig. 2(b) illustrates another rock type in this area, that of massive sandstone beds. The topography consists of flat, blocky upland areas with steep valley walls. The soils are shallow upon the uplands. Fig. 2(c) illustrates the airphoto pattern developed in areas of flat-lying shales in the same general region. Rolling topography, intricate drainage, and erosion pattern indicate residual soils over shales. Here the topography is too rolling to locate an airport.

Procedure.—To provide a well-organized analysis of an area for airfield planning or any similar project, the engineer must proceed in a systematic manner. This means starting with the known and working towards the unknown:

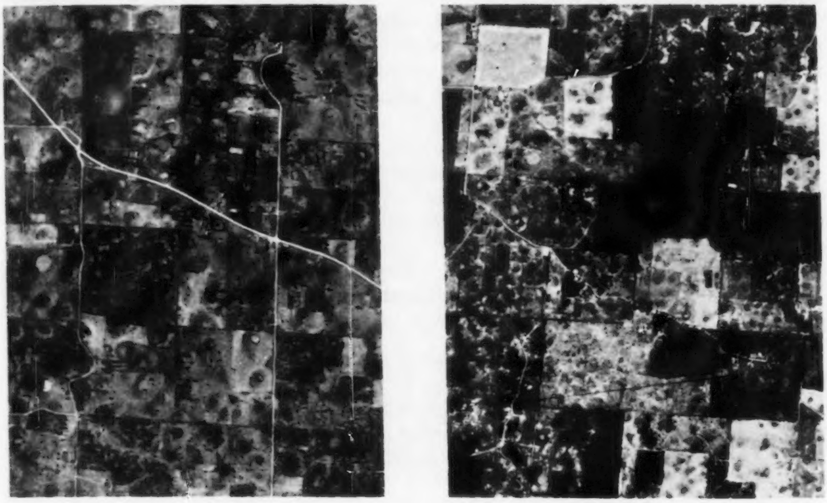

FIG. 1.—SIMILAR MATERIALS CREATE SIMILAR PATTERNS

FIG. 2.—THREE CONTRASTING AIRPHOTO PATTERNS

that is, working from the general to the particular. Photo interpretation requires the ability to select the significant details from a vast array of details. The engineer doing the analysis or interpretation should have an adequate background in soils, geology, and other earth sciences to enable him to understand a sequence of events or to evaluate the significance of features that individually or as a part of a system provide the information required to make a correct analysis.

Briefly, the logical procedure to follow is: (1) assemble a mosaic of an area somewhat larger than the area of concern, (2) study this mosaic and delineate the major natural and cultural boundaries of the landscape, (3) study stereoscopically the individual prints to determine the details within the larger areas already delineated, and (4) prepare soil and drainage maps and prepare a report on the conditions which affect the cost and the design of the project.

Engineering Use of Information.—Airphoto interpretation will save time and provide better planning in several ways. Several uses of the techniques are:

1. Preliminary interpretation of soils and other natural environmental factors that influence the planning and location of a project.
2. Engineering soils maps of proposed locations.
3. Location of construction materials.
4. Determination of potential engineering problems, such as hard rock cuts and unstable soil conditions.

It should be emphasized that, to obtain the most from airphoto interpretation, it should be utilized at the earliest stages of planning. Airphotos provide the engineer the opportunity for an areal concept of large areas during the planning stages.

AREA ANALYSIS

To illustrate the application of the technique to airfield engineering an area has been selected and an engineering analysis will be prepared for this area.

A study has been made of an area around Martinsburg, W. Va., for the purpose of selecting a suitable site for an airport to serve that city. This study has been based on interpretation of aerial photography taken in 1938 for the United States Department of Agriculture. The area studied is $8\frac{1}{2}$ miles east to west by $11\frac{1}{2}$ miles north to south with Martinsburg near the center.

Regional Considerations.—

Geography.—Martinsburg, the county seat of Berkeley County, W. Va., is in the east center of the county within the Great Valley that runs northeast through Virginia, West Virginia, and into Pennsylvania. Berkeley County has an area of approximately 192 sq miles and a population of approximately 30,000 (1950). Martinsburg, with approximately 15,000, contains most of this population. The airphotos indicate that the city is largely residential in character and the surrounding area is devoted to agriculture. Orchards cover large areas. Limestone is quarried at many places, probably for use as agricultural lime and cement.

Physiography.—The area of study lies within the Great Valley of the Ridge and Valley Province of the Appalachian Highlands. Fig. 3 illustrates the local physiography. The area is bordered on the northwest by a single sharp-crested mountain ridge, M. This gives way immediately to a broad plain, which varies

FIG. 3.—PHYSIOGRAPHIC SUBDIVISIONS OF THE AREA STUDIED

in character from a plain with hills and sharply rolling topography, Ph, to a plain that is nearly level to undulating, P. Throughout this plain a well-defined linear pattern prevails. Bordering the Shenandoah river valley, this plain becomes dissected by tributary erosion. This area, Pd-V, is transected by many deep tributary ravines giving it a hilly topography. The dividing line between these physiographic sections may be drawn by a study of the mosaic of the area.

Geology.—A detailed description of the geology of this area would be very complicated due to the complex geological history of the Ridge and Valley Province. The airphotos provide the answer to local geology. In Fig. 4 Sh indicates shale, Ss denotes sandstone, Ls indicates limestone, and Sh/Ss denotes shale over sandstone. The sharp crest and the steep east-facing slope of the ridge in the northwest corner of the area indicate that this ridge was formed by a resistant sandstone. The bedrock of the plains is composed of steeply tilted limestone and shales. The strong linear pattern of the topography indicates that the bedrock is tilted. That tilted limestone is the predominant bedrock is indicated by the linear pattern of the many sinkholes in Fig. 5. Many of the beds are being quarried for the limestone; a quarry is shown in the upper right corner of Fig. 5. Shale is indicated in areas of more softly rounded topography in the plains section and becomes predominant in the dissected plains section, as evidenced by the absence of linearity and sinkholes. The well-developed surface-drainage pattern is typical of a shale area.

The soils overlying the plains will be predominantly silty clay in texture. In some areas the soil will be very shallow; in others it will probably be 10 ft to 15 ft deep.

Climate.—Heavy woods of the uncultivated areas are an indicator of the subhumid to humid climate of the area.

Local Pattern Features.—The pattern features of each section are discussed to illustrate the method of evaluating and interpreting the soil and terrain conditions.

Mountain Section.

Landform.—A single sharp-crested ridge trending northeast-southwest. The east-facing slope is a steep escarpment facing onto the plains to the east while the western slope is more gradual with a somewhat rounded appearance. Fig. 6 illustrates the typical landform of this ridge.

Drainage.—Close parallel drainage gullies extend up the western slope. The eastern slope has no drainage system because of its steepness.

Erosional Features.—Heavy vegetation prevents study of the erosion.

Photo Tones.—The vegetation obscures the soil tones except for scattered areas on the east slope where small patches show white to light gray tones.

Vegetation.—Hardwood forest covers both slopes of the ridge, the west slope having a very dense cover. The cover on the east slope is not as dense and shows some lineal stratification.

Cultural Features.—This ridge is too steep for man to impose upon it many cultural features. Widely separated roads cross this ridge with long grades running slightly diagonal to the trend of the ridge.

Analysis.—This ridge standing high above the adjacent plain with its sharp crest and bold, steep eastern slope is typical of a tilted sandstone formation interbedded with other sedimentary rocks. The soft slopes on the west indicate that shale will be found covering the sandstone on this slope.

Plains with Hills Section.

FIG. 4.—REGIONAL GEOLOGY MAP OF AREA STUDIED

Landform.—This area consists of a plain at the base of the previously cited mountain ridges. This plain has an undulating surface with many low, rounded parallel ridges trending northeast-southwest, which gives the airphoto pattern a definite linear appearance. This linear pattern is modified in areas by the subtle features of sinkholes or solution basins. Fig. 7 is characteristic of the area; note the low parallel hills and the sinkholes in the upper left corner.

Drainage.—No surface drainage system exists. Everywhere the drainage is into small inclosed basins.

Erosional Features.—Gullies are nonexistent. Sheet erosion is undoubtedly predominant.

Photo Tones.—The photo tones vary from white to moderately dark gray. The tone changes are due to both the vegetation and the topography. The steep

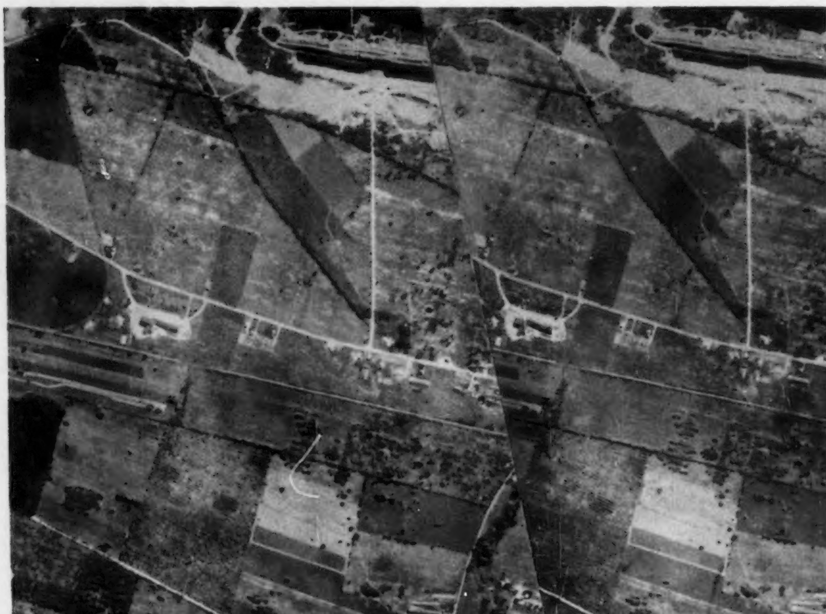

FIG. 5.—STEREOPAIR OF AREA WITHIN THE PLAINS SECTION

slopes are usually light with lower areas light to dark gray. Small irregular circular areas of dark gray indicate sinkholes or infiltration basins.

Cultural Features.—Except on the steepest slopes the area is intensively farmed. Orchards cover a large portion of the area. Field shapes are usually long and narrow, accenting the linear features of the topography.

Special Features.—The strong linear features and the infiltration basins are a unique combination and each presents strong clues as to the nature of the underlying bedrock.

Analysis.—The linear pattern exhibited by the low parallel ridges indicates that this area is underlain by tilted bedrock. The roundness of the crests

and slopes along with the development of small solution basins and shallow sinkholes are characteristic of tilted limestone formations. The lack of development of a surface drainage system bears out this conclusion.

Plains Section.

Landform.—This area consists of level to moderately rolling plains. Low ridges have a parallel trend. Fig. 8 illustrates the terrain features of a nearly level area of the plains. Occasional bare rock outcrops exist at the higher elevations as shown in Fig. 5.

Drainage.—There is no surface drainage system developed except near the eastern edge where the tributaries to the Shenandoah River have reached into the plains. Shallow sinholes exist in many areas.

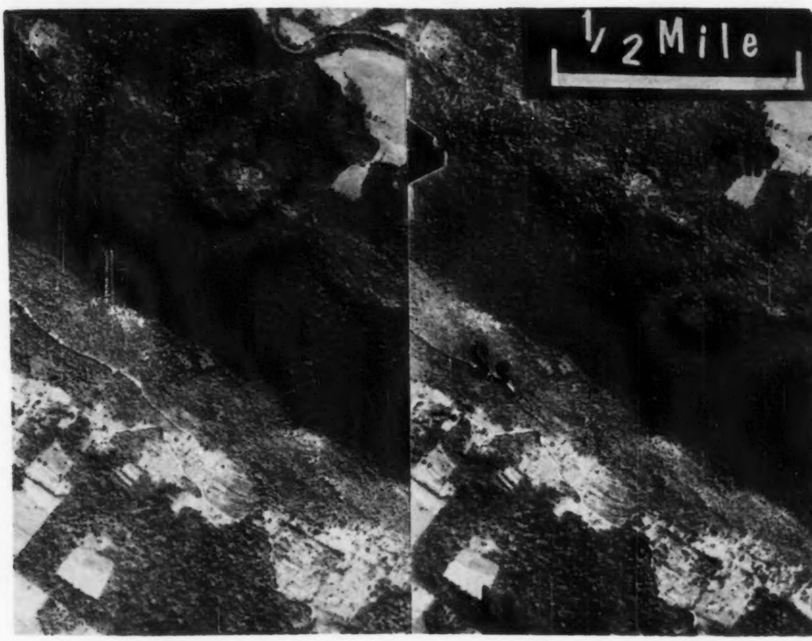

FIG. 6.—AIRPHOTO STEREOPAIR OF SANDSTONE RIDGE

Erosional Features.—Shallow broad gullies are found occasionally. Erosion is predominantly sheet erosion.

Photo Tones.—The photo tones over this plain are most strongly controlled by the crops in the fields, but the tones vary from white to gray. These tones indicate the well drained condition of the soil. Infiltration basins or sinkholes are indicated by dark gray circular areas.

Vegetation.—The present vegetation consists of grain crops and orchards as the area is intensively farmed.

Cultural Features.—The area is intensively farmed. The main roads run parallel to the trend of the shallow ridges. Fields are larger and more rectangular than in the other areas. A northeast-southwest trend, however, is still evident.

Special Features.—Within the area are large quarries that are long, narrow, and deep. These quarries evidently follow a single stratum. These quarries show no refuse pile of stone, indicating that the material is used for agricultural lime or Portland cement.

Analysis.—The low rounded hills with their distinct parallel linear trend and the evidence of sinkholes throughout the area indicate that this area is also underlain by limestone. Quarries in the area also bear out this analysis.

Plains to Valley Section.

Landform.—This area consists of a dissected plain leaving a hilly area bordering the Shenandoah River. The hills mainly trend east-west as they are formed by deep tributary ravines of the river.

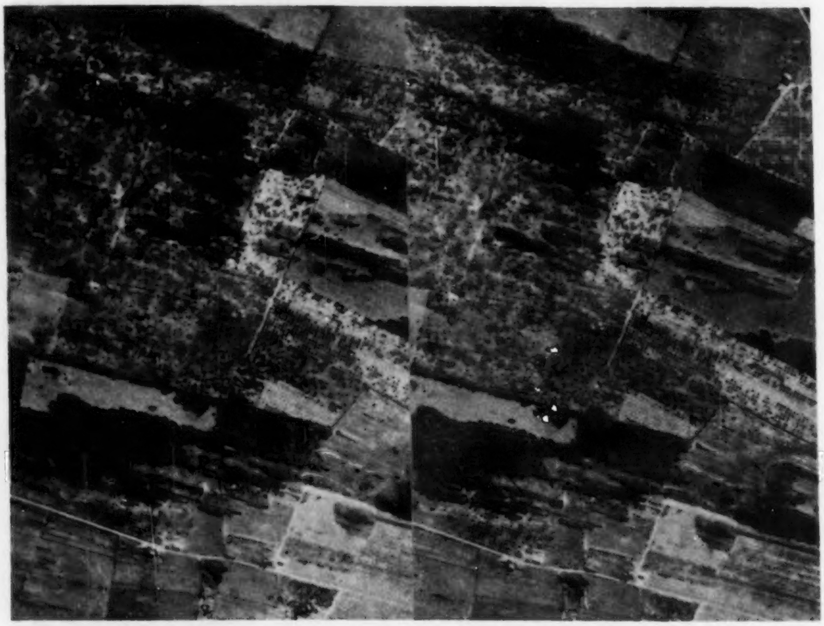

FIG. 7.—AIRPHOTO STEREOPAIR OF AREA WITHIN THE PLAINS WITH HILLS SECTION

Drainage.—The predominant drainage consists of deep ravines trending east to the Shenandoah River. A secondary drainage system exists at nearly right angles to this system, giving the overall drainage pattern a trellis appearance as illustrated in Fig. 9.

Erosional Features.—Gullies vary from rounded shapes to deep V-shape indicating differences in soils in the area. Often a gully will broaden out into a sinkhole at its head.

Photo Tones.—Throughout this area photo tones are a darker gray than in other areas.

Vegetation.—The area is farmed with fields of grain crops except in the more extremely dissected areas where hardwood forests stand.

Cultural Features.—The area is farmed in small irregular fields. Roads are strongly controlled by the topography and follow the ridge tops.

Special Features.—Within this area there is a brick plant and a quarry, indicating the nature of the material.

Analysis.—This section is composed of a plains dissected to the stage that it is a region of soft rounded hills. A well-developed trellis drainage system indicates that internal drainage is not the main source of drainage. The underlying rock is more impervious than the other areas. This all indicates that the area is underlain by interbedded sedimentaries in which shale is the predominant member.

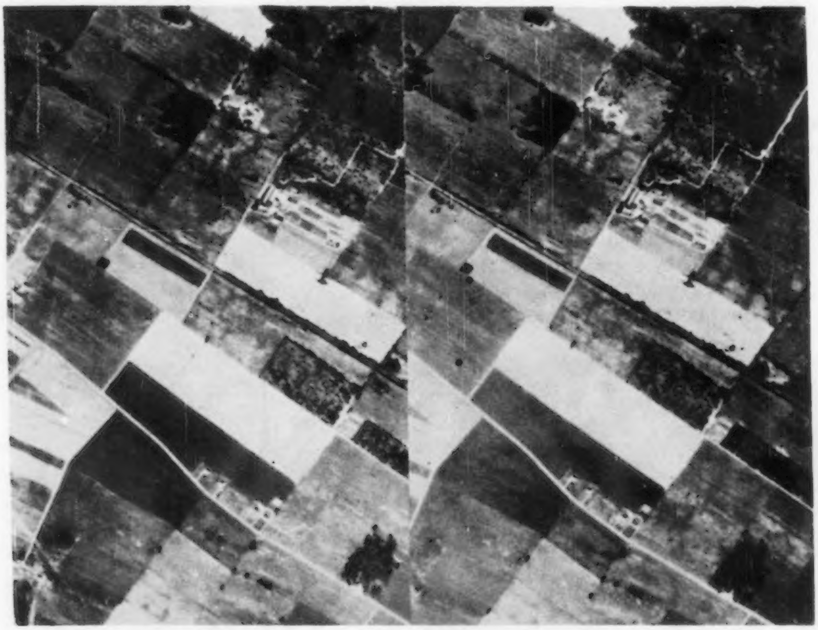

FIG. 8.—AIRPHOTO STEREOPAIR OF NEARLY LEVEL AREA OF PLAINS SECTION

Engineering Analysis and Appraisal.—Throughout the area the topography presents a strong parallel linear trend. The mountains, the low ridges of the plains area all trend northeast-southwest. Throughout much of the area the topography has too much relief to provide an adequate site for an airport. Any airport must be located far enough east of the high ridge so that the ridge does not become an approach obstacle. Only in the plains section is the terrain level enough for location of an airport.

The soils of the plains are residual soils developed from the underlying limestone. They are predominantly a plastic clay or silty clay with good internal drainage in the undisturbed state. These soils will be impervious upon being remolded during construction. The depth of these soils will vary. Many areas where bedrock outcrops or is near the surface are found in studying the photos. In other places the soils will be 10 ft to 15 ft deep.

The soils of the dissected area near the river are probably residual soils developed over interbedded limestone and shale. These soils are not well drained internally.

In the undisturbed state, the soils of the plains are well drained internally. The sinkholes may be a problem for some construction. Drainage ditches should be provided to remove water from the edges of any runways as these soils will no longer have good internal drainage and are apt to swell and lose strength if the surface water is not drained away from the area.

Embankments for runways will have to be constructed from the local residual soils. Base course and paving materials can be obtained from the limestone quarries in the area.

APPLICATION OF INFORMATION TO AIRFIELD SITE SELECTION

Site Selection Criteria.—The following criteria were established before the airphoto study of sites was begun. These do not, of course, establish the need or the ability of the community to support an airport.

1. Close proximity to the city served.
2. Moderately level terrain to reduce cost of construction.
3. Low-cost land.
4. No nearby approach obstacles.
5. Runways 5,000 ft long.
6. Best possible soil and drainage conditions.

FIG. 9.—AIRPHOTO AND DRAINAGE MAP

Site Selection.—From the study of the airphotos three sites were selected for preliminary study. The locations of these three sites are shown in Fig. 10.

Site 1.—This site is situated about 3 miles north of Martinsburg just west of U. S. Highway 11 and has good access by U. S. 11. The site is bounded by U. S. 11 on the east, State Highway 9 along the southwest, a railroad track running northwest-southeast along the north side and a railroad track running north-south along the west side.

The terrain is moderately to sharply rolling with local relief being approximately 20 ft. This terrain would require considerable cut and fill for runways.

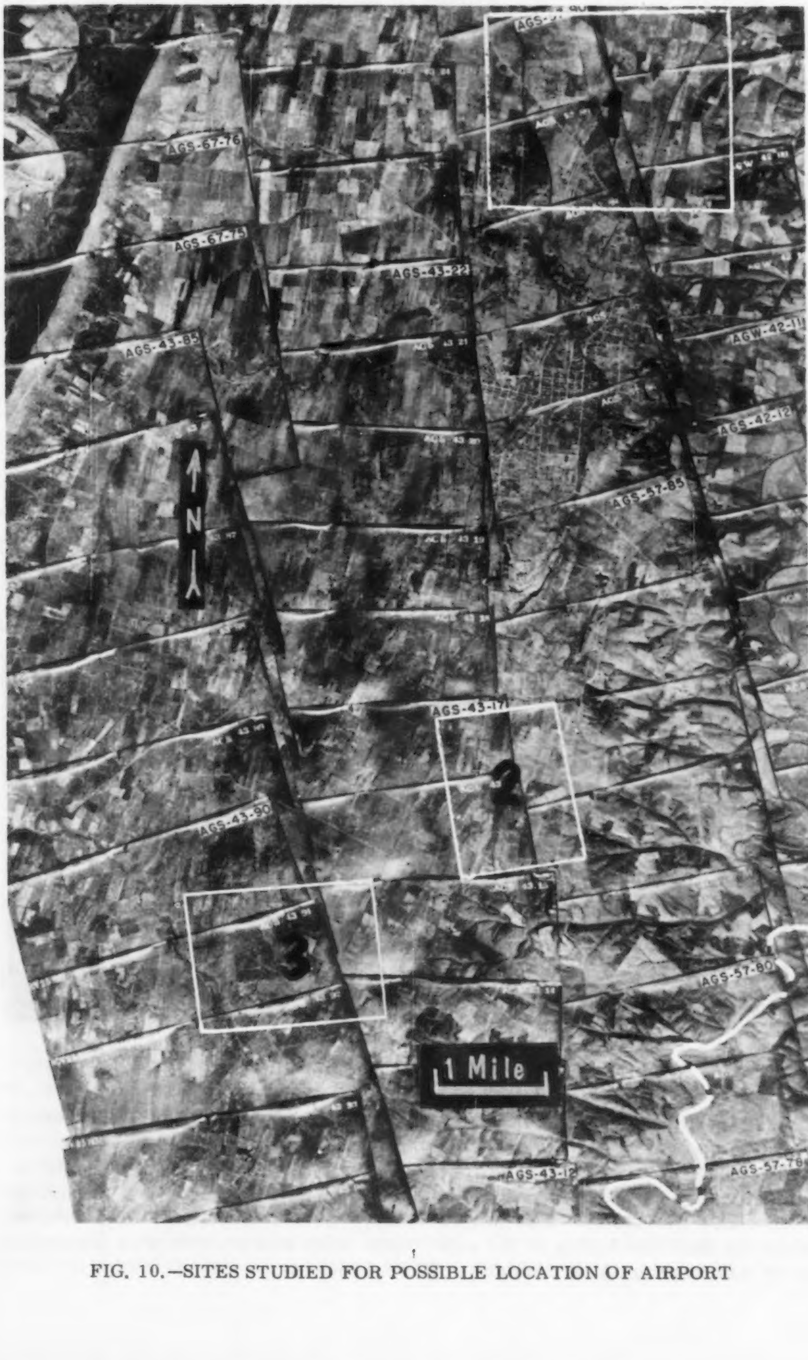

FIG. 10.—SITES STUDIED FOR POSSIBLE LOCATION OF AIRPORT

The soils of this site are residual soils developed from limestone bedrock. The airphoto indicates that the bedrock is close to the surface in many of the high areas, which means many rock cuts during construction (Fig. 11).

The site is limited by being bounded on all sides by the highways and the railroad, and has another major drawback in its close proximity to the high mountain ridge.

Site 2.—Site 2 (Fig. 12) is located approximately 3 miles south of the city just east of U. S. 11. The east-west width of the site is limited by the highway and the dissected hilly terrain to the east. The site has good access from the city by U. S. 11.

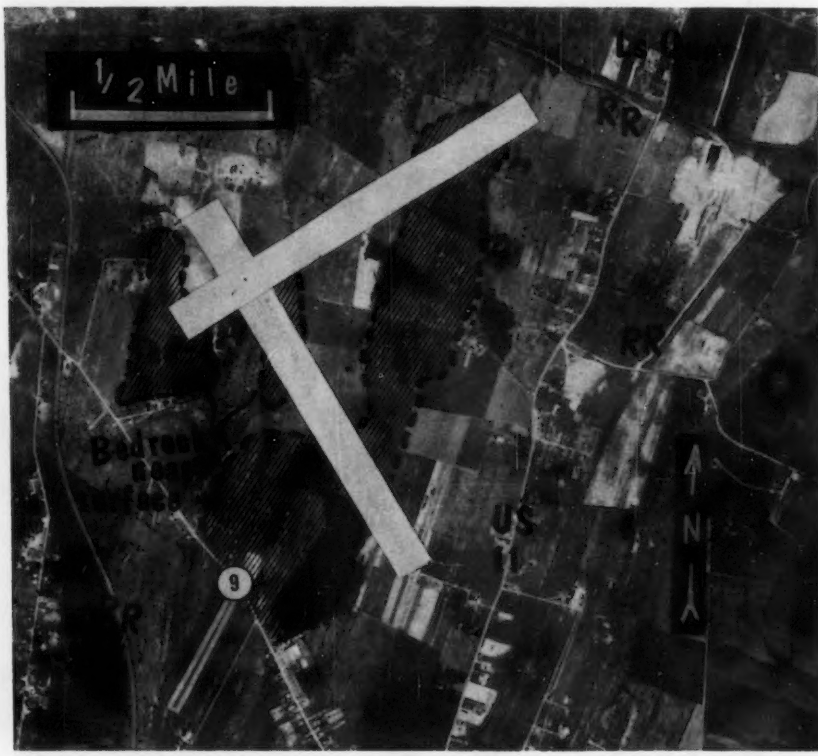

FIG. 11.—SITE MAP FOR SITE 1

The terrain is rolling and will require some deep cuts for the runways. There is no evidence, though, that bedrock is near the surface.

The soils in this site vary from plastic silty clay to clay residual soils developed over limestone to residual silty clay soils developed over shales and interbedded limestone. The soils developed over shale are predominant and cover the east two-thirds of the site. These soils will provide poor subgrades which will require thick base and sub-base courses.

A drainage ditch should be provided along the south edge of the southeast leg of the runways as the runway transects a natural drainage course. A drainage map of the site is shown in Fig. 13.

A low ridge transects the ends of both runways and will require excavating to establish runway grades. The soil, however, appears to be deep in this area and rock cut will probably not be extensive.

Site 3.—Site 3 is located approximately 6 miles southwest of the city just west of U. S. 11, which would provide good access (Fig. 14).

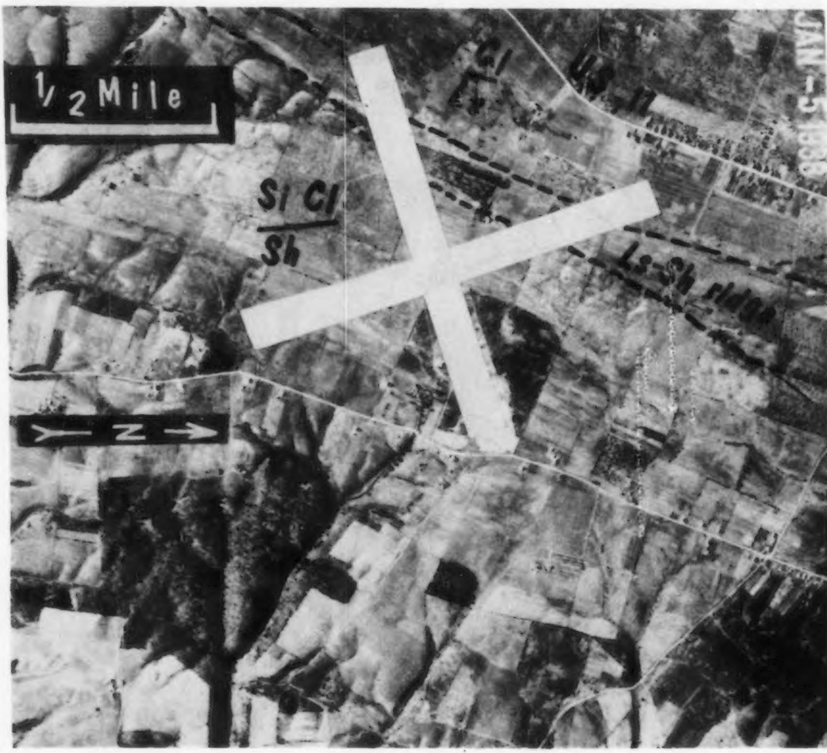

FIG. 12.—SITE MAP FOR PROPOSED SITE 2

Over the south one-half of the site the terrain is only slightly rolling. In the north half the site has considerable more relief with low hills rising 30 ft or more. Much of the local relief is created by the sinkholes and solution basins.

The soils in the area are residual soils developed from the underlying limestone. The soils are very shallow over a large part of the area and hard rock cuts would be required to provide a runway. There are many sinkholes in the area which would provide another problem. These may require grouting or bridging wherever they would occur under a runway.

The present use of the area consists of farming, with three orchards in the area required for runways.

Comparative Analysis.—Before the final selection, field exploration should be made at each site and land appraisals made to provide an adequate cost estimate of each site.

A comparison of the sites from the airphoto study indicates that Site 2 is probably the best of the three sites. Sites 1 and 3 have rock near the surface and rock cuts would be required. The undulating topography of Site 1 would require considerable earth movement to establish runway grades. Over most of Site 3 the topography is quite level but the low hills in the north part will require deep rock cuts. The sinkholes found over much of Site 3 would add a considerable cost to construction. The topography at Site 2 controls the directions of the runways and cuts will have to be made to establish a runway grade. There is, however, evidence to indicate that the soils are deeper in

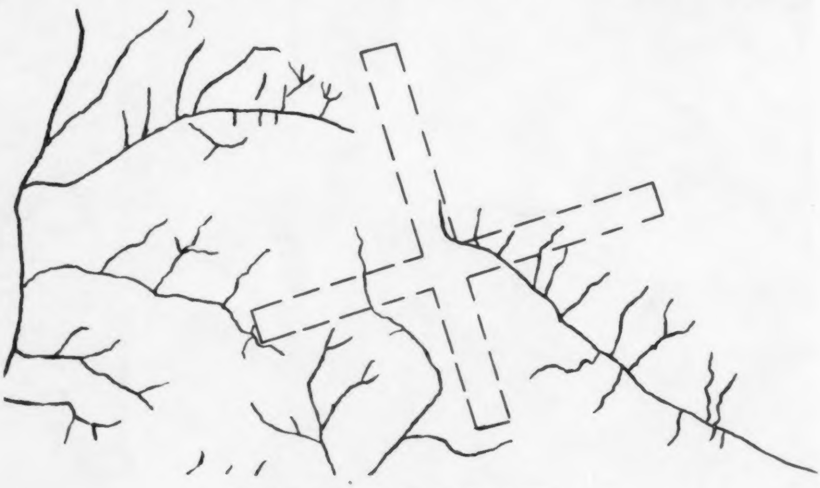

FIG. 13.—DRAINAGE MAP OF SITE 2

this area and rock cuts would not be so extensive. Sites 1 and 3 are much nearer the mountain ridge to the west, which may provide an approach obstacle.

All three sites have the same general type of soil, but as stated the soil is deeper at Site 2 than at the other sites.

Access is good to all sites, with U. S. 11 running near each one.

If land costs at all sites are nearly equal, Site 2 because of the lack of extensive rock cuts, will probably provide the most economical site that will serve the needs of the community.

SUMMARY

This report illustrates the use of airphoto interpretation in airport-site selection. The information used to select the sites could be obtained in many ways. The use of topographic maps, geology, and field data would develop the required information. Airphoto interpretation gives the engineer a perspective

that none of these other items can. He is able to make a comprehensive evaluation of all the environmental factors that enter into the analysis.

It is not suggested that the airphotos will provide all the answers, but then neither does a soil auger, or a geology report. The airphoto is a tool that should be used with all the other means of obtaining information. It should not supplant field investigation. Instead it should be used to plan and as an aid to field explorations. Its use should start at the first planning stage and continue hand in hand with all other planning and investigations.

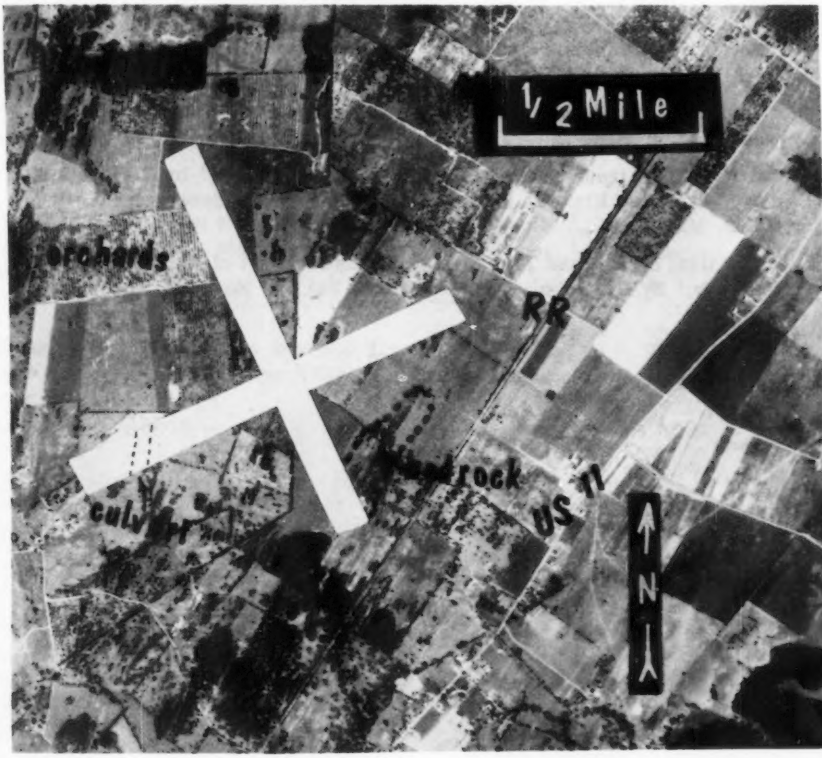

FIG. 14.—SITE MAP FOR SITE 3

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Journal of the
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Proceedings of the American Society of Civil Engineers

CONTENTS

DISCUSSION

Note.—This paper is a part of the copyrighted Journal of the Air Transport Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. AT 1, May, 1960.
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LONGITUDINAL DISTRIBUTION OF WHEEL LOADS ON A RUNWAY[a]

Discussion by R. G. Ahlvin

R. G. AHLVIN.[1]—The authors are to be congratulated for their treatment of a topic which is significant in airfield pavement design and on which a report has not previously been published. The paper is very interesting and worthy of study by those concerned with airfield pavements.

Since the United States Corps of Engineers', Dept of the Army, design requirements are used for some of the authors' comparisons, a few comments on the development of those requirements may be in order. The 10% reduction in thickness was adopted by the Corps of Engineers for portions of runways between the 1,000-ft ends because of failures that had been experienced in the ends of constant-thickness runways.

Following recognition of this failure trend, studies similar to those reported in this paper were made in attempts to determine why the ends of runways should fail earlier than the center sections. It was concluded that wing lift, increasing with increase in velocity, while obviously a contributing factor, did not completely account for the runway-end failure phenomenon.

The authors, in their final paragraphs, recognize a factor other than lift which plays an important part in design thickness requirements—the relation between "static" and "moving" loadings. Tests conducted[2,3] by the Corps of Engineers at Marietta, Georgia, in 1945, and at Stockton, Calif., during 1945-1947 clearly indicated significantly smaller stresses and deflections beneath slowly moving loads than beneath static loads.

The Corps of Engineers' design thickness reductions for interiors of runways are based on the combined effect of lift and moving loadings. At the time the reductions were originally incorporated in the design, a 20% reduction was considered, but only a 10% reduction was adopted. Thus, for propeller-type aircraft, the 10% thickness reductions are somewhat conservative. The reported study, in showing jet aircraft to have a lesser portion of their weight airborne 1,000 ft from the runway end, indicates that the 10% thickness reduction is less conservative for jet aircraft than for propeller aircraft. It does not indicate an unconservative element in current design criteria.

In a number of instances, airfields constructed by the Corps of Engineers have been subjected to significant amounts of traffic by heavy jet aircraft.

---

[a] October, 1959, by R. W. Smith and R. Horonjeff.

[1] U. S. Army Corps of Engrs., Waterways Experiment Sta., Civ. Engrg., Vicksburg, Miss.

[2] "Certain Requirements for Flexible Pavement Designs for B-29 Planes," U. S. Corps of Engrs., Waterways Experiment Sta., Tech. Memorandum, August, 1945.

[3] "Accelerated Traffic Test at Stockton Airfield, Stockton, Calif.," U. S. Corps of Engrs., Sacramento Dist., (Stockton Test No. 2), May, 1948.

Condition surveys have in no case indicated pavement distress initiating in the second 1,000 ft of runway on any of these airfields.

The reported study does point the way to possible further reductions in thickness in the central portion of very long runways. This possibility has already been recognized and is not being ignored.

As an example of progress in effecting economies in regard to traffic distribution, there is another traffic factor which can be considered in the construction of large airfields for heavy aircraft. Recent Corps of Engineers' criteria for very heavy load airfields have taken advantage of this additional consideration. It concerns the lateral distribution of traffic. Pavements for occasional use can be significantly thinner than those for regular use, and the lateral distribution of aircraft on a wide runway is such that strips of the runway one-third of its width and along each edge are rarely, if ever, loaded by using aircraft. For heavy aircraft, the reduced thicknesses in these edge strips are still adequate for service and snow removal equipment, and current Corps of Engineers' criteria permit reduced strength designs in the 100-ft-edge strips of 300-ft heavy-duty runways. R. Horonjeff, in joint effort with J. H. Jones, reported[4] a study of this nature.[4]

4 "The Effect of Traffic upon Runway Pavement Cross Section," by R. Horonjeff and John Hugh Jones, Proceedings, ASCE, June, 1955.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbols (PP). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper number are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 2270 is identified as 2270(ST9) which indicates that the paper is contained in the ninth issue of the Journal of the Structural Division during 1959.

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MAY: 2014(AT2), 2015(AT2), 2016(AT2), 2017(HY5), 2018(HY5), 2019(HY5), 2020(HY5), 2021(HY5), 2022(HY5), 2023(PL2), 2024(PL2), 2025(PL2), 2026(PP1), 2027(PP1), 2028(PP1), 2029(PP1), 2030(SA3), 2031(SA3), 2032(SA3), 2033(SA3), 2034(ST5), 2035(ST5), 2036(ST5), 2037(ST5), 2038(PL2), 2039(PL2), 2040(AT2)[c], 2041(PL2)[c], 2042(PP1)[c], 2043(ST5)[c], 2044(SA3)[c], 2045(HY5)[c], 2046(PP1), 2047(PP1).

JUNE: 2048(CP1), 2049(CP1), 2050(CP1), 2051(CP1), 2052(CP1), 2053(CP1), 2054(CP1), 2055(CP1), 2056 (HY6), 2057(HY6), 2058(HY6), 2059(IR2), 2060(IR2), 2061(PO3), 2062(SM3), 2063(SM3), 2064(SM3), 2065 (ST6), 2066(WW2), 2067(WW2), 2068(WW2), 2069(WW2), 2070(WW2), 2071(WW2), 2072(CP1)[c], 2073(IR2)[c], 2074(PO3)[c], 2075(ST6)[c], 2076(HY6)[c], 2077(SM3)[c], 2078(WW2)[c].

JULY: 2079(HY7), 2080(HY7), 2081(HY7), 2082(HY7), 2083(HY7), 2084(HY7), 2085(HY7), 2086(SA4), 2087 (SA4), 2088(SA4), 2089(SA4), 2090(SA4), 2091(EM3), 2092(EM3), 2093(EM3), 2094(EM3), 2095(EM3), 2096 (EM3), 2097(HY7)[c], 2098(SA4)[c], 2099(EM3)[c], 2100(AT3), 2101(AT3), 2102(AT3), 2103(AT3), 2104(AT3), 2105(AT3), 2106(AT3), 2107(AT3), 2108(AT3), 2109(AT3), 2110(AT3), 2111(AT3), 2112(AT3), 2113(AT3), 2114(AT3), 2115(AT3), 2116(AT3), 2117(AT3), 2118(AT3), 2119(AT3), 2120(AT3), 2121(AT3), 2122(AT3), 2123(AT3), 2124(AT3), 2125(AT3).

AUGUST: 2126(HY8), 2127(HY8), 2128(HY8), 2129(HY8), 2130(PO4), 2131(PO4), 2132(PO4), 2133(PO4), 2134 (SM4), 2135(SM4), 2136(SM4), 2137(SM4), 2138(HY8)[c], 2139(PO4)[c] 2140(SM4)[c].

SEPTEMBER: 2141(CO2), 2142(CO2), 2143(CO2), 2144(HW3), 2145(HW3), 2146(HW3), 2147(HY9), 2148(HY9), 2149(HY9), 2150(HY9), 2151(IR3), 2152(ST7)[c], 2153(IR3), 2154(IR3), 2155(IR3), 2156(IR3), 2157(IR3), 2158 (IR3), 2159(IR3), 2160(IR3), 2161(SA5), 2162(SA5), 2163(ST7), 2164(ST7), 2165(SU1), 2166(SU1), 2167(WW3), 2168(WW3), 2169(WW3), 2170(WW3), 2171(WW3), 2172(WW3), 2173(WW3), 2174(WW3), 2175(WW3), 2176 (WW3), 2177(WW3), 2178(CO2)[c], 2179(IR3)[c], 2180(HW3)[c], 2181(SA5)[c], 2182(HY9)[c], 2183(SU1)[c], 2184 (WW3)[c], 2185(PP2)[c], 2186(ST7)[c], 2187(PP2), 2188(PP2).

OCTOBER: 2189(AT4), 2190(AT4), 2191(AT4), 2192(AT4), 2193(AT4), 2194(EM4), 2195(EM4), 2196(EM4), 2197(EM4), 2198(EM4), 2199(EM4), 2200(HY10), 2201(HY10), 2202(HY10), 2203(PL3), 2204(PL3), 2205 (PL3), 2206(PO5), 2207(PO5), 2208(PO5), 2209(PO5), 2210(SM5), 2211(SM5), 2212(SM5), 2213(SM5), 2214 (SM5), 2215(SM5), 2216(SM5), 2217(SM5), 2218(ST8), 2219(ST8), 2220(EM4), 2221(ST8), 2222(ST8), 2223 (ST8), 2224(HY10), 2225(HY10), 2226(PO5), 2227(PO5), 2228(PO5), 2229(ST8), 2230(EM4), 2231(EM4), 2232(AT4)[c], 2233(PL3)[c], 2234(EM4)[c], 2235(HY10)[c], 2236(SM5)[c], 2237(ST8)[c], 2238(PO5)[c], 2239(ST8), 2240 (PL3).

NOVEMBER: 2241(HY11), 2242(HY11), 2243(HY11), 2244(HY11), 2245(HY11), 2246(SA6), 2247(SA6), 2248 (SA6), 2249(SA6), 2250(SA6), 2251(SA6), 2252(SA6), 2253(SA6), 2254(SA6), 2255(SA6), 2256(ST9), 2257(ST9), 2258(ST9), 2259(ST9), 2260(HY11), 2261(ST9)[c], 2262(ST9), 2263(HY11), 2264(ST9), 2265(HY11), 2266(SA6), 2267(SA6), 2268(SA6), 2269(HY11)[c], 2270(ST9).

DECEMBER: 2271(HY12)[c], 2272(CP2), 2273(HW4), 2274(HW4), 2275(HW4), 2276(HW4), 2277(HW4), 2278 (HW4), 2279(HW4), 2280(HW4), 2281(IR4), 2282(IR4), 2283(IR4), 2284(IR4), 2285(PO6), 2286(PO6), 2287 (PO6), 2288(PO6), 2289(PO6), 2290(PO6), 2291(PO6), 2292(SM6), 2293(SM6), 2294(SM6), 2295(SM6), 2296 (SM6), 2297(WW4), 2298(WW4), 2299(WW4), 2300(WW4), 2301(WW4), 2302(WW4), 2303(WW4), 2304(HW4), 2305(ST10), 2306(CP2), 2307(CP2), 2308(ST10), 2309(CP2), 2310(HY12), 2311(HY12), 2312(PO6), 2313(PO6), 2314(ST10), 2315(HY12), 2316(HY12), 2317(HY12), 2318(WW4), 2319(SM6), 2320(SM6), 2321(ST10), 2322 (ST10), 2323(HW4)[c], 2324(CP2)[c], 2325(SM6)[c], 2326(WW4)[c], 2327(IR4)[c], 2328(PO6)[c], 2329(ST10)[c], 2330 (CP2).

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JANUARY: 2331(EM1), 2332(EM1), 2333(EM1), 2334(EM1), 2335(HY1), 2336(HY1), 2337(EM1), 2338(EM1), 2339(HY1), 2340(HY1), 2341(SA1), 2342(EM1), 2343(SA1), 2344(ST1), 2345(ST1), 2346(ST1), 2347(ST1), 2348(EM1)[c], 2349(HY1)[c], 2350(ST1), 2351(ST1), 2352(SA1)[c], 2353(ST1)[c], 2354(ST1).

FEBRUARY: 2355(CO1), 2356(CO1), 2357(CO1), 2358(CO1), 2359(CO1), 2360(CO1), 2361(PO1), 2362(HY2), 2363(ST2), 2364(HY2), 2365(SU1), 2366(HY2), 2367(SU1), 2368(SM1), 2369(HY2), 2370(SU1), 2371(HY2), 2372(PO1), 2373(SM1), 2374(HY2), 2375(PO1), 2376(HY2), 2377(CO1)[c], 2378(SU1), 2379(SU1), 2380(SU1), 2381(HY2)[c], 2382(ST2), 2383(SU1), 2384(ST2), 2385(SU1)[c], 2386(SU1), 2387(SU1), 2388(SU1), 2389(SM1), 2390(ST2)[c], 2391(SM1)[c], 2392(PO1)[c].

MARCH: 2393(IR1), 2394(IR1), 2395(IR1), 2396(IR1), 2397(IR1), 2398(IR1), 2399(IR1), 2400(IR1), 2401(IR1), 2402(IR1), 2403(IR1), 2404(IR1), 2405(IR1), 2406(IR1), 2407(SA2), 2408(SA2), 2409(HY3), 2410(ST3), 2411 (SA2), 2412(HW1), 2413(WW1), 2414(WW1), 2415(HY3), 2416(HW1), 2417(HW3), 2418(HW1)[c], 2419(WW1)[c], 2420(WW1), 2421(WW1), 2422(WW1), 2423(WW1), 2424(SA2), 2425(SA2)[c], 2426(HY3)[c], 2427(ST3)[c].

APRIL: 2428(ST4), 2429(HY4), 2430(PO2), 2431(SM2), 2432(PO2), 2433(ST4), 2434(EM2), 2435(PO2), 2436 (ST4), 2437(ST4), 2438(HY4), 2439(EM2), 2440(EM2), 2441(ST4), 2442(SM2), 2443(HY4), 2444(ST4), 2445 (EM2), 2446(ST4), 2447(EM2), 2448(SM2), 2449(HY4), 2450(ST4), 2451(HY4), 2452(HY4), 2453(EM2), 2454 (EM2), 2455(EM2)[c], 2456(HY4)[c], 2457(PO2)[c], 2458(ST4)[c], 2459(SM2)[c].

MAY: 2460(AT1), 2461(ST5), 2462(AT1), 2463(AT1), 2464(CP1), 2465(CP1), 2466(AT1), 2467(AT1), 2468(SA3), 2469(HY5), 2470(ST5), 2471(SA3), 2472(SA3), 2473(ST5), 2474(SA3), 2475(ST5), 2476(SA3), 2477(ST5), 2478 (HY5), 2479(SA3), 2480(ST5), 2481(SA3), 2482(CO2), 2483(CO2), 2484(HY5), 2485(HY5), 2486(AT1)[c], 2487 (CP1)[c], 2488(CO2)[c], 2489(HY5)[c], 2490(SA3)[c], 2491(ST5)[c], 2492(CP1), 2493(CO2).

c. Discussion of several papers, grouped by divisions.

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